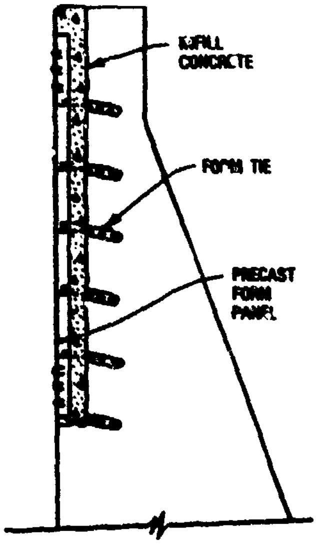
US Army Corps of Engineers

REHABILITATION RESEARCH PROGRAM

TECHNICAL REPORT REMR-CS-7

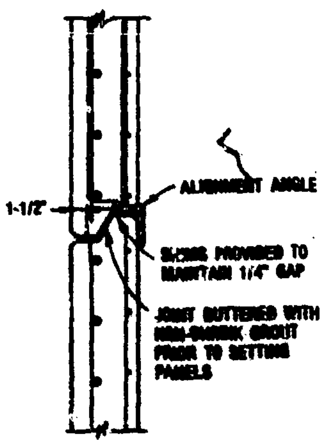
AD-A185 081

DESIGN OF A PRECAST CONCRETE STAY-IN-PLACE FORMING SYSTEM FOR LOCK WALL REHABILITATION



by

ABAM Engineers, Inc.
33301 Ninth Avenue South
Federal Way, Washington 98003-6395



SECTION
HORIZONTAL JOINT

July 1987

Final Report

Approved For Public Release; Distribution Unlimited

Prepared for DEPARTMENT OF THE ARMY
US Army Corps of Engineers
Washington, DC 20314-1000

Under Contract No. DACW39-86-C-0014
(Civil Works Research Work Unit 32273)

Monitored by Structures Laboratory
US Army Engineer Waterways Experiment Station
PO Box 631, Vicksburg, Mississippi 39180-0631

87

The following two letters used as part of the number designating technical reports of research published under the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program identify the problem area under which the report was prepared:

	Problem Area		Problem Area
CS	Concrete and Steel Structures	EM	Electrical and Mechanical
GT	Geotechnical	EI	Environmental Impacts
HY	Hydraulics	OM	Operations Management
CO	Coastal		

For example, Technical Report REMR-CS-7 is the seventh report published under the Concrete and Steel Structures problem area.

COVER PHOTOS:

TOP — Precast concrete stay-in-place forming system for lock wall rehabilitation.

BOTTOM — Typical horizontal joint between precast panels.

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE

REPORT DOCUMENTATION PAGE

1a. REPORT SECURITY CLASSIFICATION
Unclassified

1b. RESTRICTIVE MARKINGS

2a. SECURITY CLASSIFICATION AUTHORITY

2b. DECLASSIFICATION / DOWNGRADING SCHEDULE

3. DISTRIBUTION / AVAILABILITY OF REPORT
Available for public release; distribution unlimited.

4. PERFORMING ORGANIZATION REPORT NUMBER(S)

5. MONITORING ORGANIZATION REPORT NUMBER(S)
Technical Report REMR-CS-7

6a. NAME OF PERFORMING ORGANIZATION
ABAM Engineers, Inc.

6b. OFFICE SYMBOL *(If applicable)*

7a. NAME OF MONITORING ORGANIZATION
USAEWES
Structures Laboratory

6c. ADDRESS *(City, State, and ZIP Code)*
33301 Ninth Avenue South
Federal Way, WA 98003-6395

7b. ADDRESS *(City, State, and ZIP Code)*
PO Box 631
Vicksburg, MS 39180-0631

8a. NAME OF FUNDING / SPONSORING ORGANIZATION
US Army Corps of Engineers

8b. OFFICE SYMBOL *(If applicable)*

9. PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER

8c. ADDRESS *(City, State, and ZIP Code)*
Washington, DC 20314-1000

10. SOURCE OF FUNDING NUMBERS

PROGRAM ELEMENT NO.	PROJECT NO.	TASK NO.	WORK UNIT ACCESSION NO.

11. TITLE *(Include Security Classification)*
Design of a Precast Concrete Stay-in-Place Forming System for Lock Wall Rehabilitation

12. PERSONAL AUTHOR(S)

13a. TYPE OF REPORT	13b. TIME COVERED	14. DATE OF REPORT (Year, Month, Day)	15. PAGE COUNT
Final Report	FROM ____ TO ____	July 1987	105

16. SUPPLEMENTARY NOTATION A report of the Concrete and Steel Structures problem area of the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program. Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161

17. COSATI CODES

FIELD	GROUP	SUB-GROUP

18. SUBJECT TERMS *(Continue on reverse if necessary and identify by block number)*
Concrete design
Lock wall rehabilitation
Precast concrete
Stay-in-place forms

19. ABSTRACT *(Continue on reverse if necessary and identify by block number)*

The general approach to lock wall rehabilitation has been to remove 1 to 3 ft of concrete from the face of the lock wall and replace it with new air-entrained concrete. One of the most persistent problems using this approach is cracking in the replacement concrete. It has been postulated that by using precast concrete as a stay-in-place form for the replacement concrete, cracking problems can be eliminated. This report describes the design of such a forming system.

A range of design alternatives was evaluated through a process of value engineering and horizontal precast panels constructed of conventional precast quality concrete were selected for detailed quantitative investigation. The panels are tied to the lock monolith along the top and bottom edges using form ties designed to support the loads of the infill concrete placement.

(Continued)

20. DISTRIBUTION / AVAILABILITY OF ABSTRACT
☒ UNCLASSIFIED/UNLIMITED ☐ SAME AS RPT. ☐ DTIC USERS

21. ABSTRACT SECURITY CLASSIFICATION
Unclassified

22a. NAME OF RESPONSIBLE INDIVIDUAL

22b. TELEPHONE *(Include Area Code)*

22c. OFFICE SYMBOL

DD FORM 1473, 84 MAR — 83 APR edition may be used until exhausted. All other editions are obsolete.

SECURITY CLASSIFICATION OF THIS PAGE
Unclassified

19. ABSTRACT (Continued).

The precast concrete stay-in-place forming system is a viable method for lock wall rehabilitation. In addition to providing a concrete surface of superior durability with minimal cracking, the estimated construction cost is about 15 percent less than conventional forming and concrete placement. Another advantage of the system is the potential reduction in the length of time that a lock must be closed to traffic during rehabilitation. With proper detailing, sequencing, and scheduling of work activities, the rehabilitation work may be accomplished with minimized impact on normal lock traffic.

PREFACE

The study reported herein was authorized by Headquarters, U.S. Army Corps of Engineers (HQUSACE), under Civil Works Research Work Unit 32273, "Rehabilitation of Navigation Locks," for which Mr. James E. McDonald is Principal Investigator. This work unit is part of the Concrete and Steel Structures Problem Area of the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program sponsored by HQUSACE. The Overview Committee at HQUSACE for the REMR Research Program consists of Messrs. James E. Crews and Bruce L. McCartney, and Dr. Tony C. Liu. Technical Monitor for this study was Dr. Liu.

The study was performed by ABAM Engineers Inc., under contract to the U.S. Army Engineer Waterways Experiment Station (WES). The contract was monitored by a Technical Review Board consisting of Dr. Liu; Mr. Thurman Gaddie, Ohio River Division; Messrs. Don Logsdon and Denny Lundberg, Rock Island District; Mr. Roy Campbell, Sr., WES; and Mr. McDonald, Chairman. Principal investigators for ABAM Engineers Inc. were Messrs. Charles W. Dolan, Donald D. Magura, David C. Koski, and Elmer W. Ozolin.

The study was conducted under the general supervision of Mr. Bryant Mather, Chief, Structures Laboratory (SL), and Mr. John W. Scanlon, Chief, Concrete Technology Division (CTD), and under the direct supervision of Mr. James E. McDonald, Research Civil Engineer (CTD), who was the Contracting Officer's Representative. Program Manager for REMR is Mr. William F. McCleese, CTD.

COL Dwayne G. Lee, CE, is Commander and Director of WES. Dr. Robert W. Whalin is Technical Director.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic metres
cubic yards	0.7645549	cubic metres
Fahrenheit degrees	5/9	Celsius degrees or kelvins*
feet	0.3048	metres
inches	25.4	millimetres
kips (force)	4.44822	kilonewtons
kip-inches	112.9848	newton-metres
kips (force) per square inch	6.894757	megapascals
kips (force) per square foot	47.88026	kilopascals
ounces (avoirdupois)	0.02834952	kilograms
pounds (force) per square inch	0.006894757	megapascals
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
pounds (mass) per cubic yard	432.49842	kilograms per cubic metre
square feet	0.09290304	square metres

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings, use $K = (5/9)(F - 32) + 273.15$.

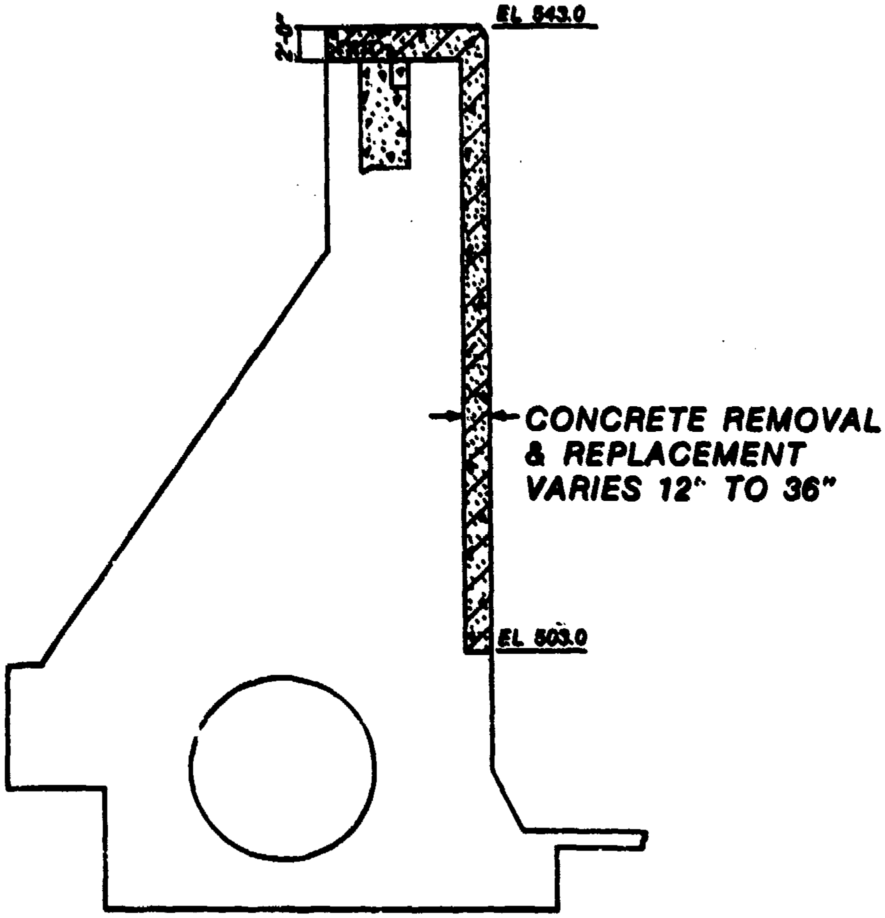
DESIGN OF A PRECAST CONCRETE STAY-IN-PLACE FORMING SYSTEM FOR LOCK WALL REHABILITATION

PART I: INTRODUCTION

Overview

1. The stay-in-place forms project is a developmental design effort which was initiated by the Corps of Engineers to evaluate the use of precast concrete as a means of controlling persistent cracking which has been experienced on previous repairs of navigation locks using conventional cast-in-place concrete and to decrease the time the lock is out of service for repairs. Conventional repairs to gravity lock structures are typically made by removing between 1 to 3 ft of concrete from the face of the lock wall and casting an overlay repair using air-entrained concrete and conventional wood or metal formwork (Figure 1). Cracking of the replacement concrete due to thermal and shrinkage strains in the new concrete overlay has consistently been a problem using this method of repair. It has been postulated that by using precast concrete as a stay-in-place form for the replacement, the thermal and shrinkage strains can be controlled such that surface cracking of the repair can be eliminated. The stay-in-place forming system may also improve the construction schedule and reduce lock closure times over conventional rehabilitation methods.

2. The Corps of Engineers currently operates and maintains 133 navigation locks which were built prior to 1940. More than 75% of these older locks are located in the Corps' North Central and Ohio River divisions, areas of relatively severe exposure to freezing and thawing. Due to the age and exposure of these structures, many exhibit significant concrete deterioration since the concrete in these structures does not contain intentionally entrained air and is therefore susceptible to damage from freezing and thawing. The extent of deterioration ranges from surface scaling to several feet in depth and is generally confined to that area of the wall above low-water pool elevation. In some



TYPICAL APPROACH TO LOCK WALL REHABILITATION

FIGURE 1

instances, however, repairs may be required 2 to 3 ft below low pool elevation.

3. Concrete removal is usually accomplished by line drilling and blasting, or by various nonexplosive methods. If required, concrete removal continues beyond the removal line until all unsound or deteriorated concrete has been removed. When the concrete wall surface preparation is completed, small-diameter holes are drilled into the face of the wall into which dowels are grouted to anchor the replacement concrete to the existing lock wall monolith. Mats of reinforcing steel are hung vertically on the dowels. In some cases, the reinforcing mat, wall armor, and other lock wall appurtenances are installed on the form prior to positioning the form on the face of the lock wall. Once the reinforcement and formwork are in position, replacement concrete is placed. Lift heights vary from 5 ft to the full wall height of approximately 40 ft. Forms are typically removed beginning one day after placement, and a membrane curing compound is applied.

4. One of the most persistent problems in lock wall rehabilitation using the conventional cast-in-place repairs is cracking in the replacement concrete. To date, the replacement concrete in all of the Corps' major rehabilitation projects has exhibited some degree of cracking. Although several variations in concrete materials, mixture proportions, and construction procedures have been used in attempts to control or eliminate the cracking, only limited success as been achieved.

Objectives

5. The objectives of the stay-in-place precast concrete form rehabilitation project are to develop a repair concept which provides superior durability, minimizes the lock closure duration, accommodates all of the normal lock hardware and appurtenances, and can be implemented at a wide variety of navigation lock sites throughout the United States. To accomplish these goals, the system must satisfy a well-defined set of durability, functional, constructability, and cost/schedule criteria. Establishing the criteria and baselines for evaluating the performance of the system is an integral part of the project and was used in value engineering analyses of the individual elements of the system.

Approach

6. The project was a two-phased effort. The first phase, which is the subject of this report, was the engineering development portion. The second phase was a demonstration project implemented following successful completion and evaluation of the Phase I work. Phase I, Concept Development, was performed in four tasks. The four Phase I tasks were organized as a filter system to maximize the number of possible options which were considered and used to test each option against a definitive set of criteria. Value engineering analyses and contractor input were used to select the final concept which optimizes the return investment. In Task 1, the project objectives and criteria were defined and quantified. In Task 2, the range of possible solutions were generated and qualitatively evaluated against the criteria. The purpose of Task 3 was to refine the preferred concept by definitive testing against the objectives, and in Task 4 the final plans, specifications, and estimates for the Phase II demonstration project were prepared.

PART II: CRITERIA

Introduction

7. A detailed set of criteria has been developed against which to evaluate each of the possible stay-in-place form concept design options. The basis of the criteria is a generic list of objectives defined by the Corps in the initial solicitation. The generic objectives for the stay-in-place precast forming system are listed below:

- a. Must be economically feasible in comparison with lock wall refacing by conventional means
- b. Must be of sufficient durability to protect the underlying concrete from further deterioration under anticipated service condition (freezing and thawing, wetting and drying, etc.)
- c. Must be anchored and fully bonded to the existing lock wall concrete
- d. Must be capable of accommodating the usual lock wall appurtenances, including wall armor, corner protection, exit ladders, line hooks, and floating mooring bitts
- e. Panel sizes must be such that they can be transported and erected using conventional equipment
- f. Must not seriously affect appearance or operation of the navigation lock
- g. For purposes of the design, a lock wall monolith width of 30 ft and a height of 40 ft to be refaced was assumed

8. From this list, specific criteria were developed which are grouped into four broad categories: durability, functionality, constructability, and cost/schedule. Each of these categories is described below.

Durability

9. The durability objectives of the repair include eliminating surface cracking, providing suitable resistance to cycles of freezing and thawing for the replacement concrete and substrates, assessing creep characteristics of prestressed options, and providing adequate cover or physical protection for reinforcing. Each of these objectives was evaluated in the design of the various precast options.

10. Differential strains between old and new concrete are a result of thermal, shrinkage, creep and, to some extent, external actions. The most significant source of strain in conventional repairs appears to be the result of both plastic and autogenous shrinkage strains in the fresh concrete repair. Thermal strains are important; however, they do not appear to be the principal cause of the persistent cracking that has been observed. The reasons for suspecting shrinkage as the primary cause is based in part on reports from the Corps of cracking appearing on repairs within 24 hours of placing the cast-in-place overlay, and even as the forms are being removed. Thermal tensile strains would not be predominant at this early age since hydration of the repair concrete would still be active. Also, cracking has appeared in structures constructed in hot weather reducing the influence of thermal "shock" during form removal.

11. Several options exist for mitigating the influence of shrinkage and thermal strains. Such options include the use of low-heat concrete mixture designs which utilize fly ash, chilled aggregates, and mixture water, or by using other admixtures designed to limit the amount or rate of heat generation within the replacement concrete. Shrinkage strains can be controlled through the use of suitable curing practices or by limiting the free surface area of the concrete overlay. Unavoidable cracking can be made more palatable by using control joints or by using reinforcing to distribute the cracks and limit crack widths.

12. Long-term durability of the repair requires that adequate resistance to cycles of freezing and thawing (the primary cause of the deterioration of lock walls) be provided in the structure. Due to the age of the structures, the concretes used in the original construction were not intentionally air entrained and were not designed to be impermeable. Consequently, the structures are susceptible to deterioration from freezing and thawing. Another aspect of this type of deterioration that must be assessed is the possibility of moisture collecting behind the repaired surface where it could freeze and form ice lenses. These ice lenses would result in debonding of the surface overlay and eventual deterioration of the surface. If free moisture can be trapped beneath the repair and if it is susceptible to freezing, then it must be allowed to drain.

13. The preferred choice for any repair to the navigation locks is to use air-entrained concrete. Entrained air has been demonstrated to be instrumental in improving resistance of concrete structures to freezing and thawing. In addition, impermeability of the mixture is important to prevent moisture penetration through the repair where it might cause continued deterioration of the substrates. Concrete protection for reinforcing also needs to be considered. Adequate cover for reinforcing must be provided to prevent corrosion.

Functionality

14. Functionality criteria for the precast forming system require that it serve to adequately support the loads imposed by the infill concrete, that it accommodate all of the normal lock hardware and appurtenances, that the panels accommodate tolerances on fabrication and erection, and that suitable details are incorporated in the panel design to resist abrasion and impact from use of the lock. Normal structural design of the forming system will account for the loads imposed on the panels. Careful attention to detailing is necessary to satisfactorily provide for fabrication and erection tolerances. Precast concrete is somewhat unforgiving when misfitting or poorly aligned panels are to be incorporated into the work. Therefore, connections and inserts must be detailed to allow for normal construction tolerances and practices.

15. For the precast system to serve as a viable alternative to conventional repairs, it must accommodate all of the lock hardware and appurtenances. In addition, panel joints must be armored where they are susceptible to impact and abrasion. Standard hardware may need to be modified, or special hardware developed to allow it to be integrated into the precast repair system.

Constructability

16. The overriding benefit to the use of precast concrete, stay-in-place forms for navigation lock repairs is in the constructability improvements that they offer. These benefits, stated as criteria for the system design, require that the system provide for maximum scheduling flexibility, that it be suitable for use at a wide

variety of construction sites and precasting facilities, that panel sizes be favorable to local transportation restrictions, that a maximum of out-of-lock preassembly be accomplished, and that special techniques and equipment usage be minimized.

17. Constructability benefits are best realized by careful design and detailing of the panels. Embedding all armor and appurtenances in the panels; designing and detailing the panels, connections, and form ties to allow for rapid erection; full-height concrete placements; interchanging of pieces; and limiting the size and weight of the panels are all important to the success of the system.

18. Industry standard practices should be employed in the panel fabrication to permit the panels to be produced locally or at remote sites. Concrete mixture designs should allow for local aggregates and materials to be used. Prestressing may be employed when plant production of the panels is possible.

Cost/Schedule

19. The criteria developed for evaluating the precast stay-in-place forming system are based on a typical 30-ft-wide by 40-ft-high concrete wall refacing. Cost and schedule data were developed from repairs performed at the Brandon Road and Lockport locks on the Illinois Waterway and from standard cost estimating and scheduling methods. The schedule criteria are considered to be of paramount importance for the design of the precast panel system, since a primary objective of the stay-in-place form repair is to reduce the lock closure duration. Cost data are used for comparison purposes only.

20. Figure 2 illustrates the baseline schedule criteria developed for a conventional cast-in-place repair. The baseline indicates that a conventional repair requires approximately 64 hours of effort to reface the typical monolith wall. The schedule duration for a large repair job, however, would be roughly one half or 32 hours per monolith due to overlapping work on adjacent monoliths as indicated by Monoliths V through W at the bottom of Figure 2. A contractor operating with experienced crews and performing shift work could be expected to reface one monolith every two to three days.

STAY-IN-PLACE FORM LOCK REHABILITATION

```
   Activity                                  Duration (hours)
                            1      1C         20        30        40        50        60        70
1. Line drill & saw cut    ==========
2. Load holes & shoot                ======
3. Clean-up & dental work                   ========
4. Drill for conc. anchors                     ===================
5. Set anchors                                      ==================
6. Set reinforcing                                              ============
7. Fabricate forms         ============
8. Set forms                                                                ========
9. Install Armor                    ============
10. Install line hooks              ==
11. Install ladders                   ====
12. Place concrete                                                                  ====
13. Strip forms & clean-up                                                              ====

                           <---------------------------64 hours--------------------------->

   ----Monolith "Y"-------><----------------------Monolith "X" Work Duration---------------><--Mon. "Z"--

   ---------------------------End Work on Monolith "W"----><----Begin Work on Monolith "Y"-------
```

Bases:
- Typical monolith segment is 30'W x 40'H
- Typical monolith segment includes ladder, line hooks, mooring bitt and armor
- Assumes 50% overlap on multiple monolith segments of lock chamber work
- Approximate progress based on 8 hour shift = 4 days per monolith segment

SCHEDULE BASELINE

FIGURE 2

21. The conventional rehabilitation projects generally involve maintenance work to ancillary lock facilities such as gates, electrical systems, or controls. With the conventional repair, this work may be accomplished concurrent with wall resurfacing work. The schedule baseline assumes that wall refacing work is pacing the construction schedule, however.

22. Table 1 illustrates the baseline cost criteria developed for a cast-in-place repair and is used for comparing relative costs of the stay-in-place form systems. The costs indicate that a conventional repair should cost on the order of $137 per square foot of lock wall surface.

PART III: PREPARATION OF ALTERNATIVES

Introduction

23. The range of possible options for achieving the project objectives was evaluated through a process of value engineering. The process involved listing the available design alernatives and then evaluating the attributes of the option against the criteria and objectives. The design alternatives which were considered can be categorized as materials related, system configuration variables, and panel types and sizes. An evaluation system was used to establish the relative preferences of each of the options which were considered. The selection of a most advantageous combination of panel design features was then made.

Material Options

24. The material options which were evaluated in the precast stay-in-place forming system design were concrete mixture variables, reinforcing options, and the possible use of surface treatments for the panels. Rehabilitation of the lock walls will likely include a minimum of two separate concrete mixture designs. One mixture will be proportioned specifically for the precast form elements with resistance (durability) to freezing and thawing, low moisture absorption qualities, abrasion resistance, and strength as predominant criteria. A second mixture for the infill concrete will be proportioned with low hydration temperatures and high workability with rapid slump loss as primary criteria. Reinforcing is provided as necessary to satisfy strength and serviceability criteria. Conventional mild steel reinforcing, prestressing, or fiber reinforcing options are considered. Surface treatments are possible means of improving the durability of the precast concrete panels. Such treatments might include surface hardeners or sealers to improve the abrasion resistance and moisture absorption qualities of the concrete.

Concretes

25. Conventional Concrete. The use of conventional concrete was assumed to represent the baseline case for evaluating material options in panel performance. Conventional concrete as defined for this evaluation consists of moderate-strength (f'_c = 4000 psi), air-entrained concrete. Such concrete is considered typical concrete that might be used for a conventional lock repair. A conventional concrete mixture, while used as the baseline for material performance, may be improved by the addition of suitable admixtures and by design and quality control as discussed below. Functionally, a conventional concrete mixture is nearly as good as some of the enhanced mixtures which were evaluated; however, its long-term durability and abrasion resistance may be somewhat less. A conventional mixture may also prove difficult to place under certain circumstances without admixtures to improve workability. However, the cost of conventional mixtures is the least of any of the mixtures evaluated.

26. Precast Quality Mixtures. Precast quality concrete is assumed to be 6500-psi compressive strength concrete. The material and batching controls necessary to achieve the higher strength will result in enhancements in its long-term durability. In particular, the permeability of the concrete will be reduced to the point that it will be virtually unaffected by cycles of freezing and thawing. With a moderate amount of entrained air, a precast quality concrete should exhibit improved durability and abrasion resistance over conventional mixtures. Admixtures may need to be introduced into the mixture to improve workability. The cost premium for precast quality concrete is low.

27. Fly Ash Concrete. The use of fly ash in the mixture is considered beneficial in controlling thermal properties of the mixture, in improving the workability of the fresh concrete, and in reducing the cost of the mixture through cement replacement. The use of fly ash in the mixture also reduces the permeability of the mixture and increases the compressive strength of the concrete at later ages.

28. Polymer Concretes. Several types of polymer concretes are available. All offer greatly improved strength and durability over typical concrete mixtures. Polymer concretes, however, have significant constructability disadvantages in relation to the typical concrete mixture types. The need to polymerize the mixture through the use of

heat or radiation provides a severe restriction in considering these materials for this application. The special equipment required for such a process severely limits the number of facilities that could fabricate panels. In addition, a system which relies upon highly specialized expertise for a key element in the construction is subject to wide schedule or price variation. The cost of polymer concrete is the primary detriment of this material. Polymer concrete mixtures might prove to be economically viable for relatively small repairs, however. Polymer mixtures were therefore evaluated in conjunction with the very thin panel types and for the bonded repair techniques.

29. Silica Fume Concrete. Silica fume added to the concrete mixture has recently been used as a cement substitute to increase the compressive strength, greatly reduce the permeability, and improve the abrasion resistance of the concrete. However, the impact resistance of the concrete may be reduced. Silica fume additives have historically been rather expensive. With the commercial introduction of silica fume additives, the cost and availability of the material should improve. Nevertheless, the use of silica fume is believed to have adverse cost and schedule implications as compared to a conventional or precast quality mixture.

30. Fiber-Reinforced Concrete. Nonmetallic fiber additives in the concrete mixtures could improve the durability of the concrete; however, the cost and constructability of the mixture would be adversely affected. Glass fiber reinforcements (GFRs) are often used to increase the tensile strength and ductility of the concrete. GFR concretes, however, are often difficult to place and are typically employed in specialized products such as pipe where these enhanced properties are especially beneficial. The long-term durability of GFR concretes has not yet been demonstrated.

Reinforcing

31. Conventional Mild Steel. Conventional mild steel reinforcing for the precast panels was used as the baseline for comparison of the various reinforcing options. Mild steel designed and detailed to satisfy the provisions of applicable codes should be adequate for strength and serviceability performance. Loads due to panel handling may require special consideration. Limiting crack widths to ensure durability of

the panels in the wet/dry environment is necessary for satisfactory long-term performance.

32. Prestressing Strand. Prestressing was evaluated as a means to accommodate construction loads due to handling the precast panels, to improve the serviceability stresses, and for crack control. The disadvantages of prestressing are the increased cost and the special facilities required to prestress the panels. On-site prestressing is highly improbable unless a mobile stressing bed were available. Therefore, prestressing was only considered as an option if required for stress and serviceability reasons.

33. Steel Fiber Reinforcing. Steel fiber reinforcing like GFRC has been used to improve the tensile strength and ductility of concretes. Like GFRC, steel fiber reinforcing is typically used in specialized applications and where the facilities are available to produce the product. The primary disadvantage of steel fiber reinforcement is the cost.

Special Treatments

34. Untreated Concrete. Untreated precast panels were considered as the base case for evaluating the merits of the treatments which were considered. Untreated panels should perform satisfactorily; however, surface treatments may offer substantial improvements to performance at a nominal cost. For this reason, surface treatments were evaluated.

35. Surface Hardeners. Several types of commercial surface treatments are available which render a very hard, abrasion-resistant finish to concrete surfaces. These treatments are typically used on floors of warehouses where heavy loads, impact, and abrasion are present. The typical treatment involves applying a powdered product to the fresh concrete finish and troweling it into the surface. Spray applications are also available. The product is either a metallic or nonmetallic powder which, when worked into the surface of the fresh concrete, creates a hard, dense, impermeable finish. Metallic hardeners tend to discolor and rust the surface when used in moist environments. The products could be applied to the inside surface of the precast panel form prior to placing the concrete in the form, thereby creating a panel with a hardened surface.

36. Surface Sealants. Surface sealants are typically spray-applied products designed to seal the surface of the concrete and prevent moisture loss. The product improves the hydration of the concrete near the surface and results in a more durable surface. Sealants also assist in preventing moisture penetration into the surface and thereby improve the resistance to freezing and thawing deterioration. The use of surface sealants to assist in curing the precast panels in the fabrication yard is a possibility. The long-term performance of surface sealants in the abrasive environment of a navigation lock is questionable.

Panel Systems and Configurations

37. Two primary panel support systems were evaluated, a system consisting of panels supported from the monolith with form ties and a system which used external bracing systems to support the panels during the infill concrete placing. A third type, consisting of bonded panels, was evaluated but is not considered to be practical for large areas of repair. With each of the support systems that was considered, the panels can be oriented either vertically or horizontally. There are then four combinations of panel support systems and configurations which were evaluated.

Tied Panel System

38. The tied panel system uses form ties to maintain the alignment of the panels and to support the loads of the infill concrete placement. The ties may also be used to replace the concrete anchors normally used in Corps lock rehabilitation work. Advantages of the tied panel system are that no major equipment is required in the lock during panel installation and concrete placing activities. The contractor also has greater flexibility in scheduling and sequencing the work. The lock may be returned to use soon after the infill concrete has been placed and consolidated. This system would allow work to be performed on an operational lock with only minor disruptions in normal lock traffic during concrete panel installation and concrete placement activities.

39. A disadvantage of the tied panel system is that it depends heavily on the ability of the ties and panel connections to accommodate

the maximum out-of-tolerance condition. If through tie holes are provided in the panels, they will require patching. Also, the maximum size of panel that may be used with this system may be governed by the capacity of the form ties. Concrete placement rates may need to be restricted to control loads on the panels or the ties.

Externally Braced System

40. The externally braced system uses a temporary support structure within the lock to support the precast panels while placing the infill concrete. The brace needs to be designed to support the full pressure of the wet concrete and may become quite substantial, depending on the height of placement. On the Brandon Road lock repair, the contractor used a falsework truss to support monolith placements on each side of the lock concurrently. The truss spanned the full width of the lock.

41. Advantages of the external bracing system are that it does not rely on the wall anchorage system, it can minimize tolerance interfaces, it can accommodate large panel sizes, and lock hardware and appurtenances can be incorporated with minimal interference. Disadvantages of the system are that major equipment is required in the lock throughout the repair, the panels must be installed between the brace and the lock wall, the normal concrete anchorage system is still required, and the cost of bracing must be included in the cost of the repair.

Vertical Arrangement

42. The vertical panel arrangement requires that full-height of panel infill concrete placements be used. Consequently, the form ties or external braces must be designed for the full-height concrete loads. Joints between panels must be match cast, armored, and of close tolerance to prevent excessive abrasion from passing barge traffic. Horizontal armor must consist of relatively short segments, making them more prone to damage from misalignment at the joints. The full-height concrete placements may result in a shorter overall construction schedule, assuming that all work is performed on an out-of-service lock.

Horizontal Arrangement

43. This arrangement has two features which enhance the panel's durability and constructability over the vertical arrangement. First, the panel joints are oriented horizontally where they will be less prone to abrasion from passing barge traffic. Horizontal armor may run the full width of the monolith and possibly be incorporated into the panel horizontal joints. Second, the ability to control infill concrete pressures with staged lifts results in a reduction in the potential for stress cracking of the panel. Staged concrete lifts may lengthen the overall construction schedule. However, the lock may continue in operation, provided a tied support system is used.

Panel Types

Hollow Core Panels

44. Hollow core precast panels are often used in building construction as floor, roof, or wall elements. They have the advantage of being relatively lightweight, fire-resistant, and economical for that application. They are produced in a slipform fashion by casting the concrete around a mandrel which moves as the panel is cast. The primary disadvantages with the use of hollow core elements for lock rehabilitation work is that the panels are not particularly durable. The cores would need to be filled, and the slipform casting process does not readily accommodate lock hardware and appurtenances.

Flat Slab Panels

45. Flat slab panels are considered to be the most likely choice for the stay-in-place forming elements. The principal advantages are the ease of production and handling and the versatility of the panels to a wide range of specific size and configuration requirements. Disadvantages include the possible need for stiffening or mechanical interlock on the interior face of the panel to anchor it to the substrate.

Double-T and Tri-Slab Panels

46. Double-T panels are considered to be a special case of the flat panel system. Backside stiffeners in the form of ribs could be added to the flat panels to assist in resisting the infill concrete

pressures. Disadvantages to this configuration include the difficulty in forming the panels and integrating the lock hardware and appurtenances into the panel. Other problems include interferences between the panel ribs and concrete anchors protruding from the monolith wall.

Thick or Thin Panels

47. The possible use of exceptionally thick or thin panels was evaluated to ascertain any potential advantage afforded by these options. Thin panel systems offer the advantage of light weight; however, they cannot accommodate lock hardware easily, and they will invariably require supplemental exterior bracing to carry the infill concrete pressures. Thin panels are viable for repair of small areas by directly bonding the small thin panels to the substrate using epoxy bonding agents; however, they are less effective if there are large variations in the depth of the wall to be repaired. Such panels could be produced from very durable materials. Thick panels have the advantage of readily accommodating lock hardware; however, they are difficult to handle and may be more costly than the thinner panel systems.

Qualitative Comparisons and Findings

48. A value engineering analysis was performed on the various material, support, configuration, and panel type options that were considered. The analysis used a qualitative comparison technique in which the relative merits of the option were evaluated against an arbitrary baseline measure of performance. A value was assigned to the option in each of the four primary criteria categories, depending on whether that option was judged to perform better or worse than the baseline in that category. The four primary criteria which were considered are durability, functionality, constructability, and cost/ schedule. The values and their assignment basis were as follows:

Value	Definition
0	Baseline performance of option against criteria
-1	Option exhibits fewer criteria attributes than the baseline option

+1	Option exhibits greater criteria attributes than the baseline option
-2	Option exhibits low degree of criteria attributes
+2	Option exhibits high degree of criteria attributes

49. The results of the value engineering comparisons are shown in Table 2. The value engineering analysis results provide an overall rating of one option in relation to another. From these results, a preferred choice of precast stay-in-place form design features can be made. The preferred panel configuration consists of a tied, flat panel constructed of precast quality concrete and oriented in a horizontal arrangement.

50. The basis for selecting this configuration for further refinement may be summarized as follows. Precast quality concrete meets the primary objective of providing sufficient durability for lock exposure and application. The tied horizontal panel arrangement provides the greatest degree of flexibility in scheduling and sequencing of the work and, potentially, may allow rehabilitation work to be performed on an operational lock. Flat panels are the obvious option of choice; however, stiffening may be required to carry infill loads.

PART IV: REFINEMENT OF ALTERNATIVES

Introduction

51. Using the preferred forming system identified in Part III, paragraphs 48 through 50, a detailed quantitative investigation was conducted to determine actual sizes of form panels, tie details, hardware details, etc. These details were then extrapolated to the scaled-down demonstration installation which is to be carried out in Phase II of the project.

Loading Criteria

52. The controlling loading for design of the forming system is the lateral pressure exerted on the formwork by the fresh infill concrete. Other secondary loadings included temporary construction and wind loadings on the erected formwork and strain loadings due to the shrinkage and thermal effects of the infill concrete.

53. The selection of the design lateral pressure affects not only panel and tie design but also impacts construction methods and cost. An iterative process was therefore used to select an allowable lateral formwork pressure which results in a workable panel design based on the criteria outlined in the next section yet does not adversely impact construction. A pressure of 1.25 ksf was selected, which corresponds to potential placement rates between 5 and 9 ft per hour based on guidelines developed by the concrete industry. Use of a pressure due to an unlimited placement rate was found to result in excessive panel thickness for even the narrowest panels. For the baseline 40-ft-high monolith, the unlimited pressure would be approximately 6.0 ksf.

54. Due to the sensitivity of the overall forming concept to the lateral formwork pressure, any reductions in pressures would result in an overall cost reduction for this repair procedure. The concrete industry guidelines used to select the design pressure are based on historical data and cover a wide range of common formwork and concrete placement practices. This project will involve very specific and controlled practices which can be directed at reducing the pressures and use of general guidelines may be overrestrictive. However, quantitative

data must be gathered in order to justify reduced design pressures and appropriate quality control measures must be instituted to assure that the specified materials and practices are followed.

Panel Design

55. The formwork panels have been sized to provide strength and serviceability consistent with their required function. As stated in Part I of this report, the elimination of surface cracking is the main thrust of this project. For this reason, serviceability criteria have controlled much of the detailed design of the panels.

56. The design procedure generally followed the requirements of ACI 318, "Building Code Requirements for Reinforced Concrete." Due to the importance of the serviceability criteria, flexural design was based on the ACI 318 Alternate Design Method - Appendix B (Working Stress Design). As a cross-check, the Strength Design Method was also used with a load factor of 1.9 applied to the lateral pressure as recommended in Corps publication ETL 1110-2-265.

57. Based on the studies described in Part III, panel widths ranging between 5 and 10 ft were deemed optimum from a constructability viewpoint. These panel widths require horizontal tie spacings at approximately one half of the width of the panel in order to maintain workable tie loads. This combination of panel width to tie spacing produces a vertical load path for distributing the lateral pressures. Therefore, panels have been designed as one-way, simply supported vertical beams. The assumed load path was confirmed by finite element computer analysis for a span length to tie spacing ratio of 1.5. Only minimal two-way load distribution was noted.

58. The principal serviceability criteria include cracking, deflection, and reinforcement cover. Design for crack-free panels without prestress results in either narrow or thick panels, each having constructability and cost impacts. The use of prestress in the vertical direction to avoid cracking is impractical due to the lack of development lengths for strands to transfer the prestress force into the panels.

59. Serviceability criteria was derived from ACI 224R, "Control of Cracking in Concrete Structures," which contains guidelines for acceptable crack widths for various exposure conditions. The design

criteria selected to provide optimum serviceability for the leave-in-place formwork includes the combination of maximum estimated crack widths of 0.010 in. due to formwork pressures and minimum concrete cover at the exposed surface of 2 in. A minimum concrete cover of 3/4 in. will be used on the interior panel surfaces.

60. Panel bulging or deflections due to the lateral infill concrete pressures were maintained within the erection tolerances. This serviceability requirement did not control the design even though cracked section properties were used in the deflection computations.

61. Erection and handling stresses were investigated assuming several lifting arrangements in order to ascertain that adequate panel strength was available in the longitudinal direction. The panels were checked for a dead load of 160 pcf plus 25% impact factor. Lifting inserts and embedments common to the tilt-up building construction industry are likely to be used for panel handling.

Form Tie Design

62. Form tie loads were calculated using the design lateral pressure in conjunction with tributary areas. The resultant tie force which incorporates the worst cumulative effect of erection and precasting tolerances can result in tie loads approximately 1.5 times the horizontal reaction. Several form tie concepts were investigated with the optimum choice being a weldable grade reinforcing bar grouted into the lock wall. The rebar tie is welded to a plate embedded in the precast panel. For the test installation, a factor of safety of 3 on the resultant working load was selected to ensure redundancy and improve reliability.

63. To simplify details, the ties were designed for tension loads only. Separate compression struts or kickers are provided to resist inward construction, wind, or tie pretension loads. Since the ties are grouted into lock walls of varying strengths, the capacity of the ties must be verified by actual field pullout testing to confirm the anticipated pullout strengths.

64. The tie system replaces the No. 6 bar dowels on 2- and 4-ft centers which have been previously used for the all-cast-in-place repair procedure. The tie system provides a permanent connection to the form panel. One other major difference between precast and cast-in-place

(CIP) repair procedures is the elimination of the reinforcing mat in the CIP infill concrete.

Details

65. Details have been developed to enable incorporation of all typical lock hardware with the stay-in-place form lock repair procedure, either by casting the hardware directly into the form panel or locating the hardware at panel joints. Additional details specific to this repair procedure have also been developed to assist with panel erection or to ensure watertightness. Panels will be cast with the exterior face down against the formwork in order to obtain a dense surface free of rock pockets and air bubbles and to obtain a rough texture top surface to bond with the infill concrete. This also enables careful positioning of the embedments prior to placing concrete.

66. Horizontal armor, due to the limitations of the form panel thicknesses, has been made more compact. Depth of continuous plates embedded into the panel have been limited to the depth of concrete cover in order to minimize impact on panel strength. Headed concrete anchors which anchor the armor behind the reinforcing mat are provided. The flexibility during handling of the revised armor may require fabrication in shorter lengths and splicing with field welds immediately prior to placing in the forms. Horizontal armor for the demonstration project has been sized full scale and will enable evaluation of the armor handling flexibility.

67. The line hook has been incorporated into the precast form panel but has been detailed so that the mooring loads are transferred into the infill concrete through bearing and use of a drag strut. Reinforcing bars have been doweled into the monolith in the immediate vicinity of the line hook and project into the postulated tension failure wedge created by the drag strut. Additional vertical bars are installed from the top of the form panel after erection. These bars also pass through the failure wedge and serve to transfer the mooring load into the monolith.

68. Check posts and horizontal corner armor are hardware items that will be located within the CIP concrete cap section at the top of the lock. Existing hardware details may be used, except that some local

areas of additional lock face may need to be removed to incorporate the posts. Existing vertical armor units may also be used. These are typically located along the lock at points with major setbacks or geometry changes where a CIP transition section may be required. The vertical armor is not intended for use at vertical panel-to-panel joints.

69. Major lock appurtenances such as ladders and floating mooring bitts are integrated into the design by using short precast panels adjacent to these items. Conventional supports and embedment details are to be used to anchor these appurtenances to the monolith.

70. Panel-to-panel joints have been detailed to aid in the erection of panels and to maintain a waterproof seal. The key feature which will assist with the erection is a tapered lap joint and alignment angle. Tolerances of the joint will limit relative horizontal displacement between form panels to approximately 1/8 in. Two different seal systems are provided. At the more frequent horizontal joint, a durable 50-durometer seal will be installed, and the weight of the form panel will compress the seal. In addition, a thin nonshrink grout layer will be provided to further seal the joint. At the less frequent vertical seals, an asphalt-impregnated, open-cell foam will be used. This type of seal is highly compressible and can tolerate more movement. The ease with which these types of seals compress will simplify the form panel erection.

71. All bottom form panels include vertical alignment screws. These hardware items enable fine-tuning the location of the lowest panel, which supports the remaining panels. Therefore, a tight tolerance at this location is important to maintaining the tolerance over the full height of the overall panel installation. At other horizontal joints, elevations will be controlled by spacers or form ties supported off the lower panel. The alignment screws have been designed to support the cumulative weight of all panels for the full height of a monolith and a portion of the dead weight of the infill concrete. It has been assumed that form ties provide no contribution to supporting vertical loads.

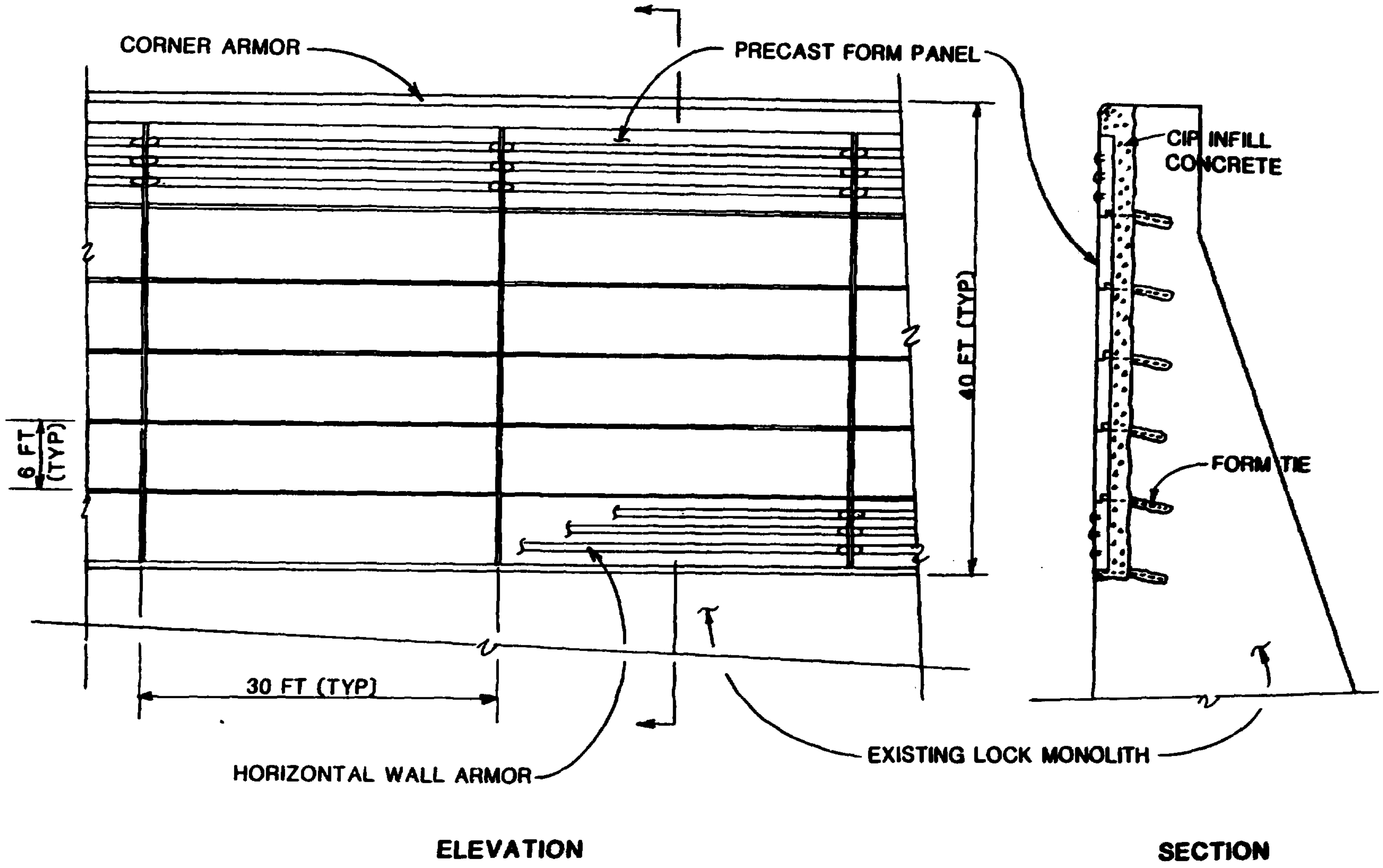
PART V: RECOMMENDATIONS, FINDINGS, AND CONCLUSIONS

System Description

72. The preferred stay-in-place forming system is illustrated in Figure 3. It consists of horizontal precast panels constructed of conventional precast quality concrete. The panels are tied to the monolith along the top and bottom edges using form ties designed to support the loads of the infill concrete placement. Infill concrete is proportioned for optimum workability and to minimize shrinkage and thermal strains.

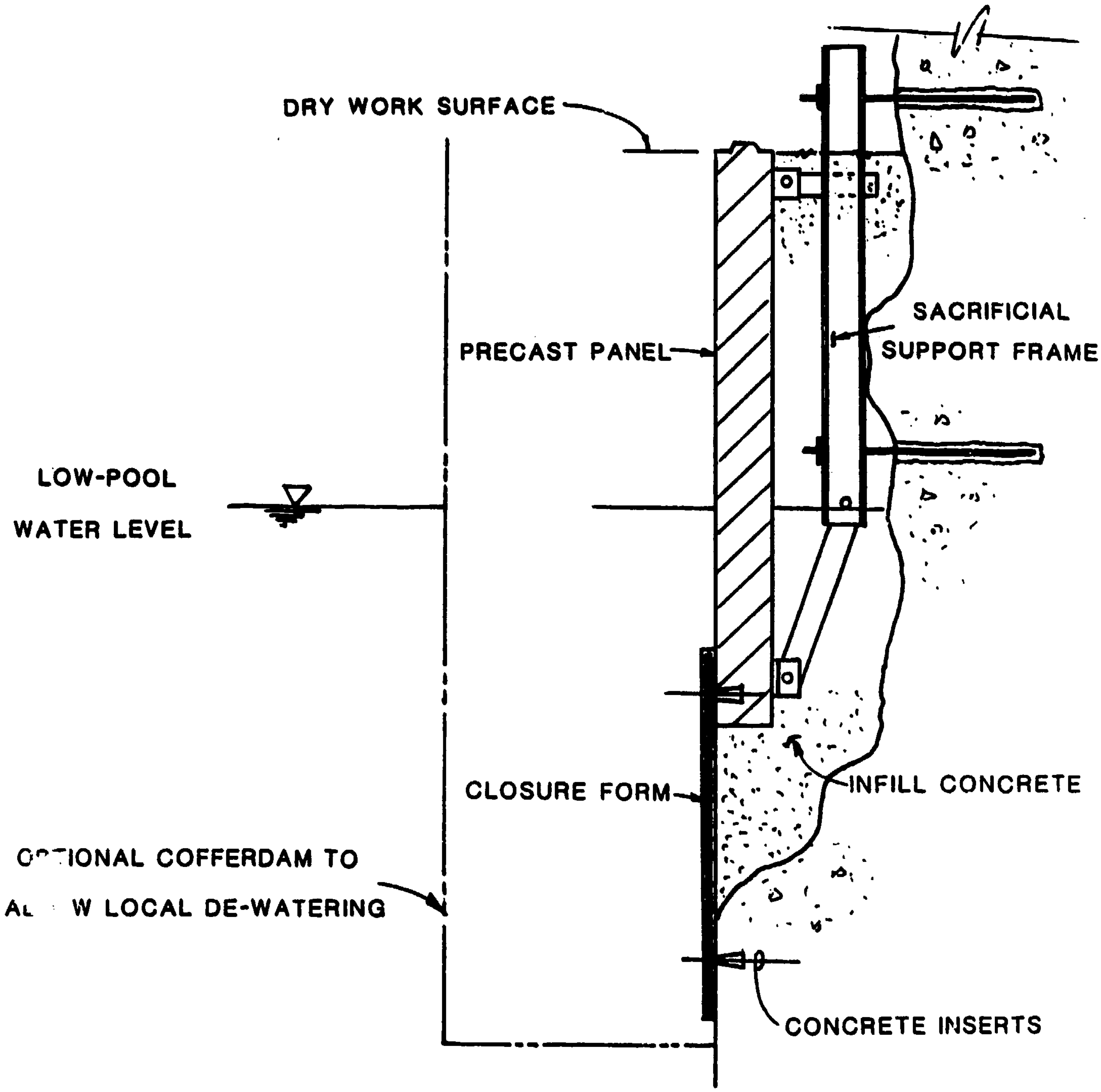
73. The panel design may be adapted to allow for work to proceed in a "wet," operational lock, but disruption of normal lock traffic would be expected. This goal may be accomplished by using special frames to support and align the bottom panel below water level while the infill concrete for the bottom lift is tremied behind the panel. Once the bottom lift has been placed, a clean work surface above low-pool water level is available for subsequent panel installation and concrete placements. A suggested detail illustrating this approach is shown in Figure 4.

74. The panels are detailed with alignment keys along their edges to facilitate installation. Placement of the bottom panel must be accomplished within tight tolerances to prevent accumulation of misalignment errors as the panels above are set. For this reason, alignment screws are provided to adjust the elevation of the bottom panel. In actual conditions, overbreakage along the bottom edge of the concrete removal area may require special details to be developed for properly aligning the bottom panel, which may have some impact on repair costs. There are numerous methods that may be used to prevent or overcome overbreakage. Figure 4 illustrates one method to accommodate overbreakage so the panels can be aligned without a uniform bearing surface. The overbreakage would be repaired as part of the infill concrete placement. It is also possible to have the closure form only to a height where a ledge can be cast that would accommodate panel installation. A third possibility is to saw cut a horizontal joint that introduces a failure plane minimizing or preventing overbreakage. This process will provide a flat bearing surface to set the lowest panel.



PRECAST STAY-IN-PLACE FORMING SYSTEM

FIGURE 3



BELOW WATER CONSTRUCTION METHODS

FIGURE 4

75. Form ties may be any commercial or specially designed product capable of carrying the loads of the infill concrete and of accommodating the construction tolerances that may be encountered. Standard manufactured form-tie products were investigated, but the lack of simple connections to the existing lock wall and precast panels and limited work space led to the selection of a welded rebar tie system. Welds to plates embedded in concrete are routinely performed in the precast concrete industry without thermally damaging the concrete. The ties must be installed snug with kickers and wedges or similar means prior to placing infill concrete. A minimum factor of safety of 3.0 on the ultimate strength of the tie is recommended for design.

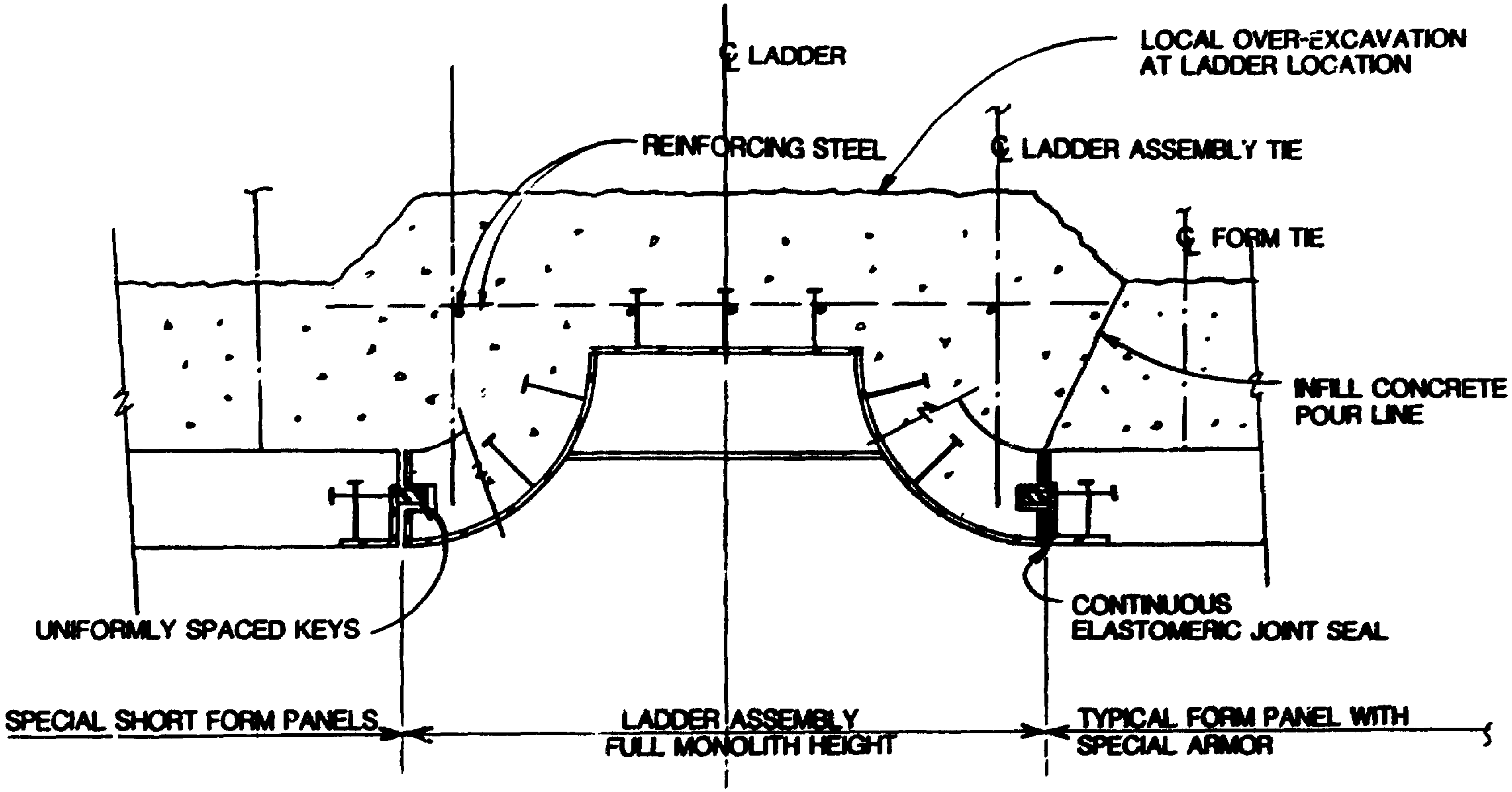
76. Panel joints should be tight fitting to prevent moisture penetration between the panels where it could freeze. The detail which has been developed utilizes compressed neoprene seals and nonshrink grout filler in the panel joint to ensure watertightness.

77. Lock hardware and appurtenances are integrated into the panel design to the maximum extent possible, resulting in the need to develop special armor hardware which will not compromise the integrity of the panels. Other standard lock hardware and appurtenances may be readily incorporated into the rehabilitation project which uses the precast system. Several details which illustrate how the precast stay-in-place panels interface with standard lock hardware and appurtenances are shown in Figures 5 and 6.

Cost Analysis

78. The estimated construction cost for lock wall rehabilitation using the precast stay-in-place forming system described above is approximately $119 per square foot of lock wall face. This cost compares with approximately $137 per square foot for conventional rehabilitation methods.

79. The costs are broken down as shown in Table 3. The cost estimates are based on an average concrete removal depth of 16 in. for a typical rehabilitation project. Also, it is assumed that some special allowance must be made to install the bottom panel below low-pool water level. The case of full-height infill-concrete placements was used for comparison purposes. It is also possible, however, to use staged

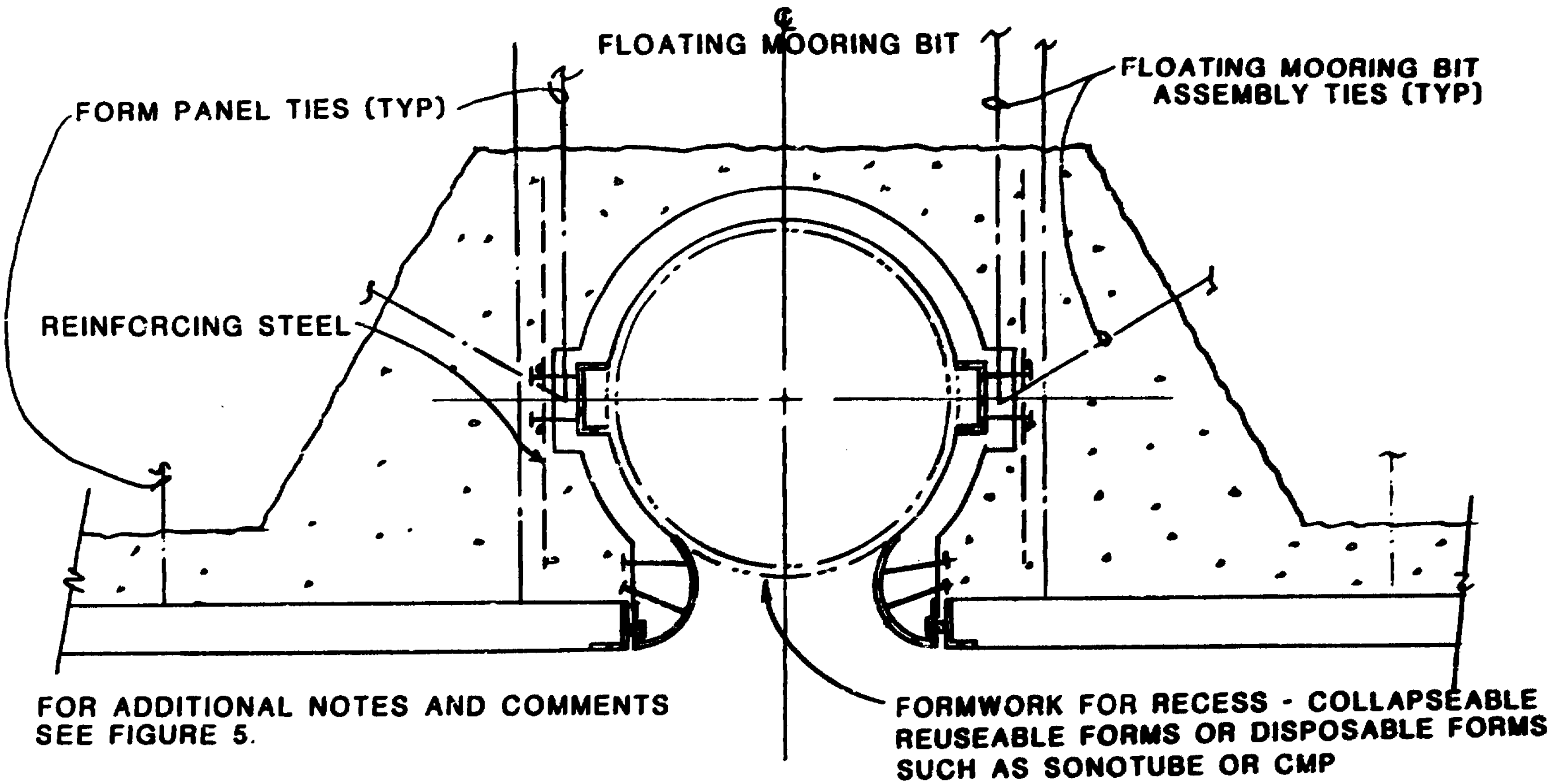


CONCEPTUAL CONSTRUCTION SEQUENCE

1. ERECT LADDER ASSEMBLY
2. INSTALL REINFORCING
3. INSTALL SPECIAL FORM PANELS TO FULL MONOLITH HEIGHT
4. CAST INFILL CONCRETE FOR ONE MONOLITH
5. ERECT FORM PANELS FOR ADJACENT MONOLITH

LOCK WALL REHABILITATION AT LADDER LOCATION

FIGURE 5



CONCEPTUAL CONSTRUCTION SEQUENCE

1. INSTALL FLOATING MOORING BIT ASSEMBLY
2. INSTALL REINFORCING STEEL
3. ERECT FORM PANELS FOR FULL MONOLITH HEIGHT
4. CAST INFILL CONCRETE

LOCK WALL REHABILITATION AT FLOATING MOORING BIT

FIGURE 6

placements. The cost estimate does not include costs attributable to loss of revenue or any other consequential costs associated with loss of use of the facility. It should be noted, however, that such costs are potentially less for the precast system since much of the work may be done with minimized impact on normal lock operations.

Schedule Analysis

80. A construction schedule assessment of the precast stay-in-place forming system is shown in Figure 7. The schedule indicates a total work duration of 30 hours to reface a typical lock wall monolith based on overlapping activities on adjacent monoliths. Work on the adjacent monolith's segments is identified as Monoliths V through W in Figure 7. This schedule compares with 32 hours estimated for the conventional repair system. However, the total time that the lock must be out of service during the use of the precast system is estimated at only 16 hours per monolith which demonstrates the significant advantage of the precast system over the conventional repair system. Restricted operation of the lock, such as limiting available widths, may also be required.

Conclusions

81. The precast concrete stay-in-place forming system is a viable method for lock wall rehabilitation. The system results in a repair of superior durability without the persistent cracking which has been experienced on conventional cast-in-place repairs. The system can accommodate typical lock hardware and armor installation and does not adversely affect the function of the repaired facility. Another advantage of the system is the potential reductions that may be achieved in the length of time that the lock must be closed to traffic. With proper detailing, sequencing, and scheduling of work activities, the rehabilitation work may be accomplished with minimized impact on normal lock traffic.

CORPS OF ENGINEERS
STAY-IN-PLACE FORM LOCK REHABILITATION

```
Activity                            Duration (hours)
                            1     10     20     30     40     50     60     70
 1. Line drill & saw cut    ==========
 2. Load holes & shoot                ======
 3. Clean-up & dental work                  ========
 4. Drill for form ties                         ============
 5. Set form ties & cure                         ========================
 6. Set panels                                                  ========
 7. Fabricate p'cast panels ================================
 8. Set forms & appurt'ces                                          ========
 9. Place concrete                                                          ====
10. Strip forms & clean-up                                                      ====

------Mon "Y"---->< ------------------------60 hours - Monolith "X"-------->< ------Mon "Z"------
----------------Mon "W"--------------------->< ----------------Mon "Y"----------------------
                                              <---16 hours--->
                                                Lock closure
```

Bases:
- Typical monolith segment is 30'W x 40'H
- Typical monolith segment includes ladder, line hooks, mooring bitt and armor
- Assumes 50% overlap on multiple monolith segments of lock chamber work
- Approximate progress based on 8 hour shift = 3.75 days per monolith segment

PRECAST SYSTEM SCHEDULE ANALYSIS

FIGURE 7

Table 1 (Rev. 1 10/1/86) * **

Cost Estimate Baseline

Item Description	Quantity	Cost Estimate Unit Cost	Cost Estimate Total
*Concrete removal	72,250 cf	$ 25.00	$1,806,250.00
**Concrete anchors	3,785 ea	48.00	181,680.00
**Reinforcing bar	174,100 lb	1.31	228,071.00
**Fabric reinforcing	2,700 sf	0.63	1,701.00
*Concrete, Type C	2,820 cy	700.00	1,974,000.00
*Concrete, Type D	850 cy	210.00	178,500.00
*Armor, government-furnished	258,100 lb	4.00	1,032,400.00
*Armor, contractor-furnished	121,900 lb	4.00	487,600.00
**Line hooks	30 ea	479.00	14,370.00
**Ladders, lock face	4 ea	6,702.00	26,808.00
Subtotal			$5,931,380.00
Engineer's estimated cost per sq ft of lock wall face			$137.05

Bases:
- o Based on Brandon Road Lock repair (December 1983)
- o Approximately 43,280 sq ft of wall surface
- o Costs do not include mobilization/demobilization, dewatering

* 1986 unit costs furnished by Rock Island District

** 1984 prices increased by 10% to 1986 prices

Table 2

Qualitative Comparisons

Concepts	Criteria: Dur-ability	Func-tional	Construc-ability	Cost/ Schedule	Overall Rating	Choice
MATERIALS						
Concretes						
Conventional concrete	0	0	0	0	0	
Precast quality mixes (6500 psi)	1	1	1	0	3	first
Fly ash concrete	1	1	1	0	3	altn
Polymer concrete	2	0	-2	-2	-2	
Silica fume concrete	2	0	1	-1	2	
Fiber-reinforced concrete	1	0	-1	-1	-1	
Reinforcing						
Conventional mild steel	0	0	0	0	0	first
Prestressing strand	1	1	-1	-1	0	altn
Steel fiber reinforcing	-1	-1	-1	-1	-4	
Special treatments						
No treatment	0	0	0	0	0	first
Surface hardeners	1	0	-1	-1	-1	
Surface sealants	1	0	-1	-1	-1	

(continued)

Table 2 (concluded)

Concepts	Criteria: Durability	Functional	Constructability	Cost/Schedule	Overall Rating	Choice
PANEL TYPES, EXTERNAL BRACED						
Vertical arrangement	-1	0	0		-1	
Horizontal arrangement	0	1	1		2	first
Hollow core	-2	-2	-2		-6	
Double-T	0	1	1		2	altn
Tri-slab	0	-1	-1		-2	
Flat slab	0	0	2		2	first
Thick panel	0	0	1		1	
Thin panel	0	-1	2		1	
Self-locking	2	0	-1		1	
Patchwork repair	unknown	unknown	unknown			
PANEL TYPES, TIED SYSTEMS						
Vertical arrangement	-1	0	1		0	
Horizontal arrangement	0	0	0		0	first
Hollow core	-2	-2	-2		-6	
Double-T	0	1	1		2	altn
Tri-slab	0	-1	-1		-2	
Flat slab	0	0	2		2	first
Thick panel	0	0	1		1	
Thin panel	0	-1	2		1	
Self-locking	2	0	-1		1	
Patchwork repair	unknown	unknown	unknown			

Table 3 (Rev. 1 10/1/86)*

Precast System Cost Analysis

Item Description	Quantity	Unit Cost	Total
* Concrete Removal	1,600 cf	$ 25.00	$ 40,000.00
Surface Preparation and Layout	1,200 sf	1.00	1,200.00
Precast Panels			
Fabricate			
Reinforcing	8,360 lb	0.90	7,524.00
* Armor	4,700 lb	4.00	18,800.00
Concrete	24 cy	1,850.00	44,400.00
Install			
* Form Ties	160 ea	50.00	8,000.00
Align and Set	8 ea	900.00	7,200.00
Cast-in-Place Concrete			
Formwork	70 sf	30.00	2,100.00
* Armor	970 lb	4.00	3,880.00
* Filler Concrete	34 cy	210.00	7,140.00
* Structural Concrete	3 cy	700.00	2,100.00
Cure	100 sf	2.00	200.00
Subtotal			$142,544.00
Estimated cost per sq ft of lock wall face			$118.79

Bases:
- o Based on a typical 30-ft wide x 40-ft high monolith
- o Assumes 16-in. average depth of concrete removed
- o Costs do not include mobilization/demobilization, dewatering
- o Costs do not include refurbishing ancillary facilities

* 1986 unit costs furnished by Rock Island District

REFERENCES

American Concrete Institute

ACI Committee 117, "Standard Tolerances for Concrete Construction and Materials," ACI 117-81, 1981, American Concrete Institute, Detroit, MI.

ACI Committee 224, "Control of Cracking in Concrete Structures," ACI 224R-80, 1980, American Concrete Institute, Detroit, MI.

ACI Committee 301, "Specification for Structural Concrete for Buildings," ACI 301-84, 1984, American Concrete Institute, Detroit, MI.

ACI Committee 318, "ACI Standard Building Code Requirements for Reinforced Concrete," ACI 318-83, 1983, American Concrete Institute, Detroit, MI.

ACI Committee 347, "Precast Concrete Units Used as Forms for Cast-in-Place Concrete," ACI 347.1R-69, 1969, American Concrete Institute, Detroit, MI.

ACI Committee 347, "Concrete Formwork," ACI 347-78, 1978 (reaffirmed 1984), American Concrete Institute, Detroit, MI.

American Concrete Institute, Designing for Effects of Creep, Shrinkage, Temperature in Concrete Structures, Publication SP-27, American Concrete Institute, Detroit, MI, 1971.

Hurd, M.K., Formwork for Concrete, Publication SP-4, American Concrete Institute, Detroit, MI, 1963.

Schrader, E., Dikeou, J., and Gill, D., "Deterioration and Repairs of Navigation Lock Concrete," Performance of Concrete in Marine Environment, Special Publication SP- 65, American Concrete Institute, Detroit, MI, 1980.

Corps of Engineers

"Dowels for Anchoring New Concrete Facing to Existing Lock Walls," ETL 1110-2-264, April 1981, U.S. Department of the Army, Corps of Engineers, Washington, DC.

"Lock Wall Accessories," ETL 1110-2-147, December 1979, U.S. Department of the Army, Corps of Engineers, Washington, DC.

"Marseilles Lock, Illinois Waterway, Refacing of Lock Walls," Project Study Report, July 1974, U.S. Army Corps of Engineers, Chicago District, Chicago, IL.

McDonald, J.E., "Rehabilitation of Lockport and Brandon Road Locks, Illinois Waterway," Memorandum for Record, WESSC, September 1984, Vicksburg, MS.

"Navigation Locks," EM 1110-2-2601, June 1959, U.S. Department of the Army, Corps of Engineers, Washington, DC.

"Planning and Design of Navigation Lock Walls and Appurtenances," EM 1110-2-2602, June 1960, U.S. Department of the Army, Corps of Engineers, Washington, DC.

"Standard Practice for Concrete," EM 1110-2-2000, September 1982, U.S. Department of the Army, Corps of Engineers, Washington, DC.

"Strength Design Criteria for Reinforced Concrete Hydraulic Structures," ETL 1110-2-265, September 1981, U.S. Department of the Army, Corps of Engineers, Washington, DC.

APPENDIX A: DESIGN CALCULATIONS
PHASE II DEMONSTRATION PROJECT

ABAM
CONSULTING ENGINEERS

Project: COE LOCK REHABILITATION STAY-IN-PLACE FORMS
Job No.: A86029
Subject: CONCRETE PRESSURES
Designer: EJO
Date: 3 July 86

DESIGN CONCRETE PRESSURE

The selection of a design pressure for the stay-in-place forms was based on a study of the interaction between concrete cover, concrete strength, panel span, and panel thickness. Subsequent to selecting the design criteria for the panels (limit crack widths to less than 0.010 in), a range of pressures and panel spans were investigated.

The results of this investigation are attached. In order to obtain a panel which has reasonable constructability attributes, a limiting pressure of 1.25 ksf was selected

Guidance for the selection of concrete formwork pressures are contained in ACI Special Publication SP-4 and ACI Committee 347 Report. Shown below are the cut-off points for pour rates based on concrete temperature.

TABLE 5-2: MAXIMUM LATERAL PRESSURE FOR DESIGN OF WALL FORMS

Based on ACI Committee 347 pressure formulas

NOTE: Do not use design pressures in excess of 2000 psf or 150 × height of fresh concrete in forms, whichever is less

Rate of placement, R, ft per hr	p, maximum lateral pressure, psf, for temperature indicated					
	90F	80F	70F	60F	50F	40F
1	250	262	278	300	330	375
2	350	375	407	450	510	600
3	450	488	536	600	690	825
4	550	600	664	750	870	1050
5	650	712	793	900	1050	1275
6	750	825	921	1050	1230	1500
7	850	938	1050	1200	1410	1725
8	881	973	1090	1246	1466	1793
9	912	1008	1130	1293	1522	1861
10	943	1043	1170	1340	1578	1929

FROM SP-4

$R \leq 7'$ $\quad p = 150 + \frac{9000\,R}{T}$ maximum = 2000 psf or 150 h, whichever is less .... (5-1)

For walls with R greater than 7 ft per hr

$$p = 150 + \frac{43{,}400}{T} + \frac{2800\,R}{T} \quad \text{maximum = 2000 psf or 150 } h\text{, whichever is less} \qquad (5\text{-}2)$$

where

p = maximum lateral pressure, psf
R = rate of placement, ft per hr
T = temperature of concrete in the forms, °F
h = maximum height of fresh concrete in the form, ft

* Although Committee 347 has recommended a formula which it believes can safely be used for form design, the scarcity of available test data limits the scope and precision of any such formula. The committee is continuing its pressure studies and has published a field test procedure to standardize and simplify the gathering of further data. Refer to "Testing Program for Lateral Pressure of Concrete," by David E. Fleming and William H. Wolf, ACI Journal, *Proceedings* V. 60, May 1963, pp. 567-574.

The above equations are based on normal 150 pcf concrete, without admixtures and typical formwork and vibration. Actual pressures for the infill concrete and stiff precast should be monitored or quantified.

ABAM
CONSULTING ENGINEERS

Sheet A3 of

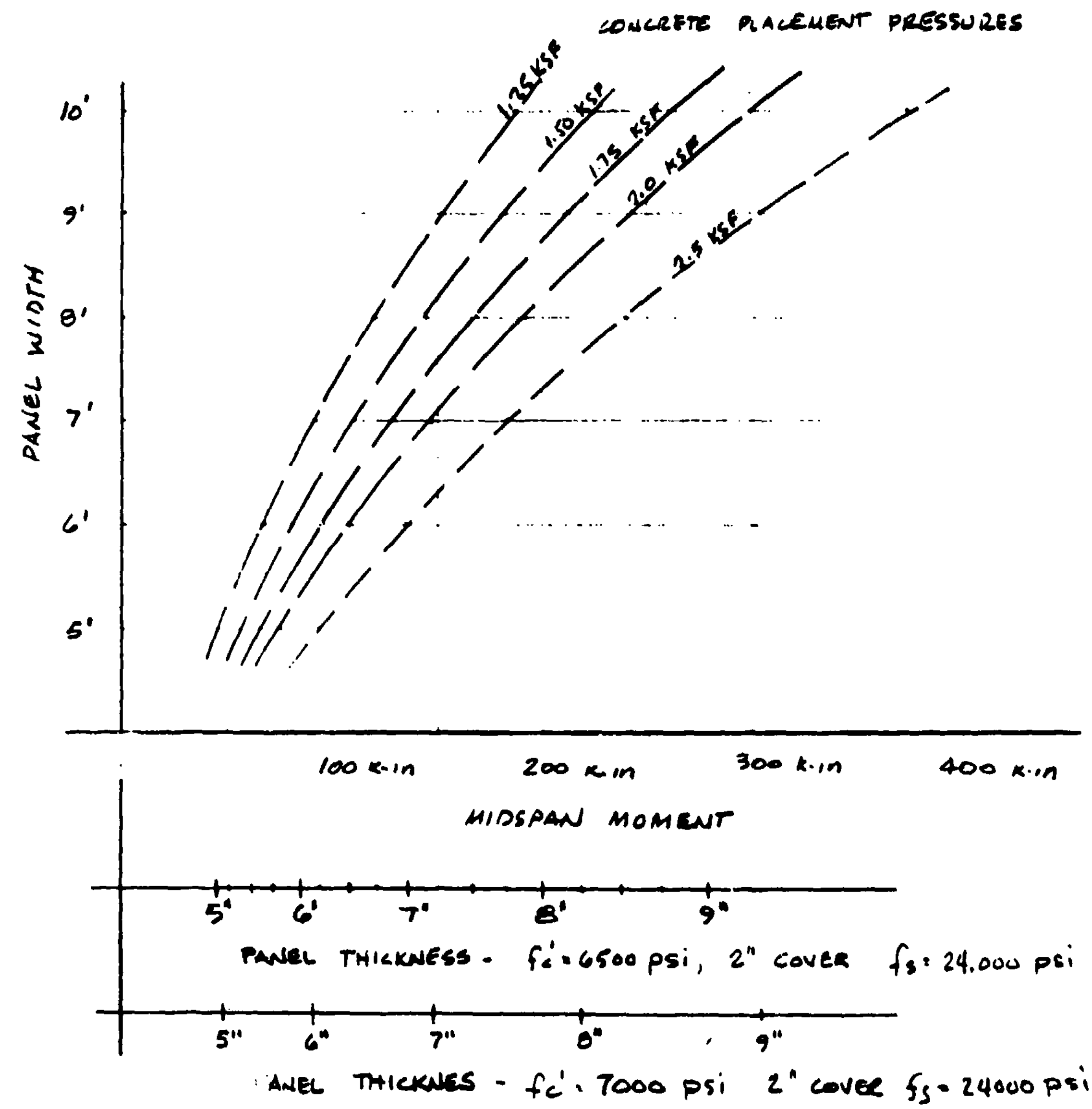
Project COE LOCK REHABILITATION
STAY-IN-PLACE FORMS

Job No. A96029

Designer EWO

Subject PANEL DESIGN FORCES

Date 11 APR 96



		Sheet	A4 of
Project	COE LOCK REHABILITATION	Job No.	A86029
	STAY-IN-PLACE FORMS	Designer	EWO
ABAM CONSULTING ENGINEERS	Subject: PANEL DESIGN	Date	21 APR 86

The Panel design will be controlled by serviceability criteria in order that cracking and long term corrosion of rebar is minimized. Also, deflections must be controlled to achieve desired erection tolerance. Cracking and deflections are determined based on working stress design. However, the design will also be verified using ultimate strength principals as appropriate.

WORKING STRESS DESIGN (ACI 318 Alternate Design Procedure). Appendix B

$$f_c = 0.45 f_c' = 0.45 \times 6500 = 2925 \text{ psi}$$

$$f_s = 24{,}000 \text{ ksi} \quad (\text{Grade 60 bars})$$

$$E_c = 33 w^{1.5} \sqrt{6500} = 33(150)^{1.5}\sqrt{6500} = 4900 \text{ ksi}$$

$$n = E_s / E_c = \frac{29{,}000}{4900} = 5.92$$

COMPUTE WORKING STRESS COEFFICIENTS

$$k = \frac{f_c}{f_c + \frac{f_s}{n}} = \frac{2925}{2925 + \frac{24000}{5.92}} = 0.4191$$

$$j = 1 - k/3 = 1 - \frac{0.4191}{3} = 0.8603$$

$$R = \tfrac{1}{2}(f_c)kj = \tfrac{1}{2} \times 2925 \times 0.4191 \times 0.8603 = 527.3 \text{ psi}$$

$$p = \frac{k}{2} \times \frac{f_c}{f_s} = \frac{0.4191}{2} \times \frac{2925}{24000} = 0.0255$$

$$M = bd^2 \times R$$

PANEL P-1 DESIGN

Design panel for full pressures which will be allowed on prototype panels: (1.25 ksf)

$$M = \frac{w\ell^2}{8} = \frac{1.25 \times 6^2}{8} \times 12 = 67.5 \text{ k-in/ft}$$

TRY PANEL h = 6.5"

$$\therefore d = 6.5" - 2" \text{ (cover)} - \tfrac{1}{2} \times \tfrac{3}{4} = 4.13 \text{ in.}$$

↑ #6 Bar

$$\text{Req'd } A_s = \frac{M}{f_s\, j d} = \frac{67.5}{24 \times 0.8603 \times 4.13} = 0.79 \text{ in}^2/\text{ft}$$

Check Steel for balanced design

$$\rho = \frac{k f_c}{2 f_s} = \frac{0.4191 \times 2925}{2 \times 24000} = 0.0255$$

$$A_{s_{req'd}} = \rho b d = 0.0255 \times 12 \times 4.13 = 1.26 \text{ in}^2/\text{ft}$$

Provide A_s based on balanced condition to keep stresses in steel lower & reduce crack widths

Use #6 Bars @ 4" OC $A_s = 1.32 \text{ in}^2/\text{ft}$

LOCATE NEUTRAL AXIS $\quad \tfrac{1}{2}(12)(x) = 5.92 \times 1.32 (4.13 - x)$

$$x = 1.76 \text{ in}$$

Check Actual Stresses

$$C = T = \frac{M}{arm} = \frac{67.5}{4.13 - \frac{1.76}{3}} = 19.07^k$$

$$f_c = \frac{19.07}{\tfrac{1}{2} \times 1.76 \times 12} = 1810 \text{ psi} \ll 2925 \text{ psi}$$

$$f_s = \frac{19.07}{1.32} = 14.45 \text{ ksi} \ll 24.0 \text{ ksi}$$

For Information, determine allowable moment based on allowable stresses

$$T = 1.32 \times 24 = 31.68^{K}$$

$$M = 31.68 \times (4.13 - \tfrac{1.76}{3}) = 112.1 \text{ k-in/ft.}$$

Verify Crack Widths

$$w = C\beta_n f_s \sqrt[3]{d_c A}$$

$$\beta_n = \frac{h_2}{h_1} = \frac{6.5-1.76}{6.5-1.76-2.375} = 2.00$$

$$A_e = (2 \times 2 + 0.75) \times 12 = 57 \text{ in}^2$$

$$m = \text{no of bars} \approx \tfrac{12}{4} = 3 \text{ bars}$$

$$A = \tfrac{57}{3} = 19.0 \text{ in}^2/\text{bar}.$$

$$w = 76 \times 10^{-6} \times 2.00 \times 14.45 \sqrt[3]{2.375 \times 19}$$

$$w \approx 0.0078 \text{ in}$$

Note : ACI 318 Commentary notes that β for one-way floor slabs ranges about 1.35

$$\text{Recheck } w = 0.0078 \times \frac{1.35}{2.00} = 0.005 \text{ in.}$$

Check Panel Stiffness

$$I_g = \tfrac{1}{12} \times 6.5^3 \times 12 = 274.6 \text{ in}^4$$

$$M_{cr} = \frac{7.5\sqrt{6500} \times 274.6}{3.25} = 51.09 \text{ k-in/ft} < 67.5 \text{ k-in/ft}$$

Panel will crack

$$\left(\frac{M_{cr}}{M_a}\right)^3 = \left(\frac{51.09}{67.5}\right)^3 = (0.757)^3 = 0.43$$

$$I_{cr} = \tfrac{1}{3} \times 1.76^3 \times 12 + 1.32 \times 5.92\,(4.13-1.77)^2 = 65.5 \text{ in}^4$$

$$I_e = 0.43 \times 274.6 + (1-0.43)(65.5) = 156.19 \text{ in}^4$$

$$\Delta = \frac{5 \times 67.5 \times 72^4}{48 \times 4900 \times 156.19} = 0.05 \text{ in.}$$ Adequate Stiffness

Check Shear Capacity

$$V_u = 1.9 \times 1.25 \times \left[\frac{6}{2} - \frac{4.13}{2}\left(\frac{1}{12}\right)\right] = 6.71^k$$

$$\overline{V_{cu}} = 0.85 \times 2\sqrt{6500} \times 4.18 \times 12 = 6.78^k > V_u \quad OK$$

Investigate Panel Handling Stresses

Assume panel will have $f_c' = 4000$ psi at time of picking from forms

$I = \frac{1}{12} \times 6.5^3 \times 12$

$I = 274.6 \text{ in}^4$

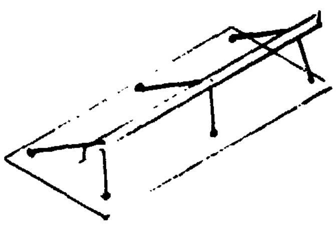
$$M_{cr} = \frac{7.5\sqrt{4000} \times 274.6}{3.25} = 40.08 \text{ k-in/ft}$$

$$= 240.5 \text{ k-in/panel}$$

Panel Weight $W = \frac{6.5}{12} \times 0.160 \times 6' = 0.52$ k/ft

2 pt pick $M = \frac{0.52 \times 30^2}{8} \times 12 = 702$ k-in/panel NG

3 pt pick $M = 702 \times \frac{15^2}{30^2} = 17.6$ k-in/panel



Appears that panel should remain uncracked during removal from forms w/ a minimum of a 3pt pick. However, the calculations have not considered impact loadings or sticking to forms during removal.

$$\text{Allowance for impact} = \frac{241}{176} = 1.37$$

Since panel will be submitted to flexure loads in longitudinal direction, provide at least min reinf.

$$\rho_{min} = \frac{200}{f_y} = \frac{200}{60,000} = 0.0033$$

Sheet A8 of	
Project COE LOCK REHABILITATION STAY-IN-PLACE FORMS	Job No. A86029
ABAM CONSULTING ENGINEERS	Designer EWD
Subject P/C PANEL DESIGN PANEL P-1	Date 28 APR 86

$A_{s\,req'd} = 0.0033 \times 4.13 \times 12 = 0.17\ in^2/ft$

#4 @ 12" oc $\quad A_s = 0.20\ in^2/ft$

However, use #4 @ 6" oc since this provides extra rebar for confining some of the embedded hardware. Also, provide light mesh at top surface since 3 point pick will put tension on each face of panel.

Note
Contractor should add any necessary bars at lifting embedments.

Use 6x6 - W2.0 x W2.0
$A_s = 0.04\ in^2/ft$

Check Panel Bending Based on Ultimate Strength.

$M_u = 1.9 \times 67.5 = 128.25\ k\text{-}in$

$M_u = \phi f_c' b d^2 \omega(1-0.59\omega)$

$\omega(1-0.59\omega) = \dfrac{128.25}{0.9 \times 6.5 \times 12 \times 4.13^2} = 0.1071$

$\omega = 0.1149$

$\rho = \omega \times \dfrac{f_c'}{f_y} = 0.1149 \times \dfrac{6.5}{60} = 0.0124$

$\rho_b = 0.85 \beta_1 \dfrac{f_c'}{f_y} \left(\dfrac{87000}{87000 + f_y} \right)$

$= 0.85 \times 0.725 \times \dfrac{6.5}{60} \times \left(\dfrac{87}{87+60} \right) = 0.0395$

$\rho_{max} = 0.75 \times 0.0395 = 0.0296$

$0.50\ \rho_b = 0.0198.$

$\rho_{used} = \dfrac{1.32}{4.13 \times 12} = 0.0266 < 0.0296 \quad OK$

$\rho_{used} > \rho = 0.0124$

Adequate Bending Ultimate Strength

ENG-001 2-84

Sheet A9 of .
Project COE LOCK REHABILITATION STAY-IN-PLACE FORMS
Job No. A56029
Designer EWP
Subject TIE FORCES
Date 14 APRIL 86

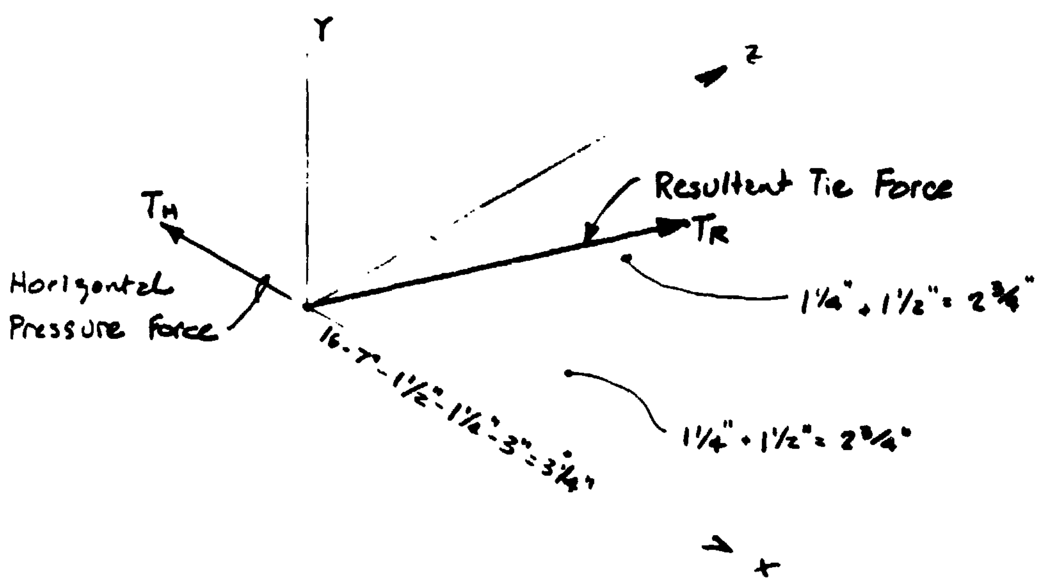
ABAM CONSULTING ENGINEERS

DETERMINE TIE FORCES (Prototype Installation)

Superimpose maximum tolerences to determine resultant tie force. See attached sketches

ASSUMPTIONS:

Panel Thickness 7"
Lockwell Excavation 16"
Tie Connection Pt ≈ 1½"
Horizontal Tie Connection Pt Tol. 1¼"
Horizontal Drill Hole Location 3"
Vertical Drill Hole Location 1½"
Vertical Tie Connection PT Tol ± 1¼"



Tie Length $= \sqrt{3.25^2 + 2.75^2 + 2.75^2} = 5.068"$

Ratio $\frac{\text{Tie Force}}{\text{Horizon Force}} = \frac{5.068}{3.25} = 1.56$

(Based on preliminary assumptions of tie connection pts)

Sheet A10 of

Project COE LOCK REHABILITATION STAY-IN PLACE FORMS

Job No. A86029

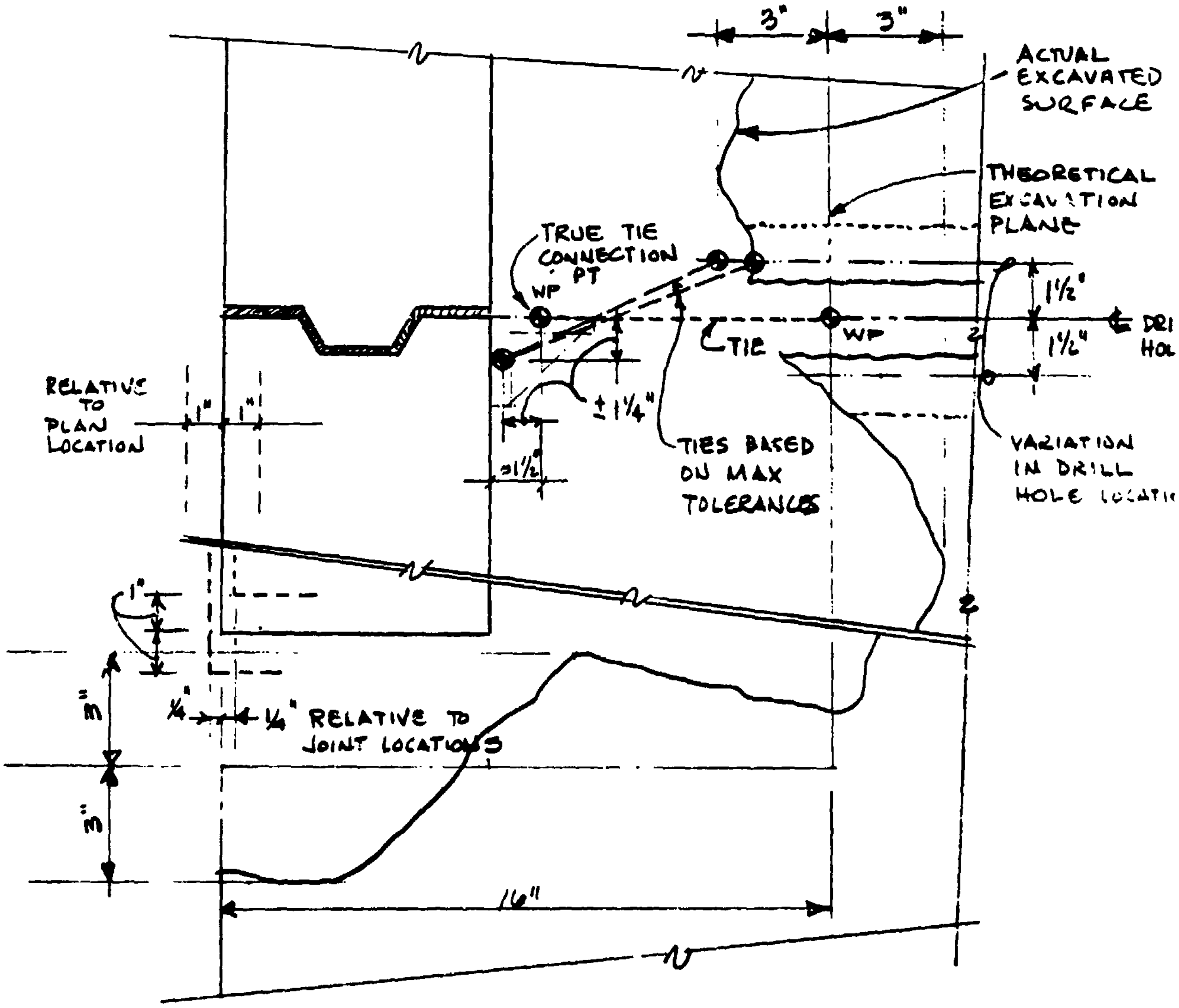
Designer EJJ

Subject TIE HARDWARE DETAILS

Date 11 APR 96

ABAM CONSULTING ENGINEERS

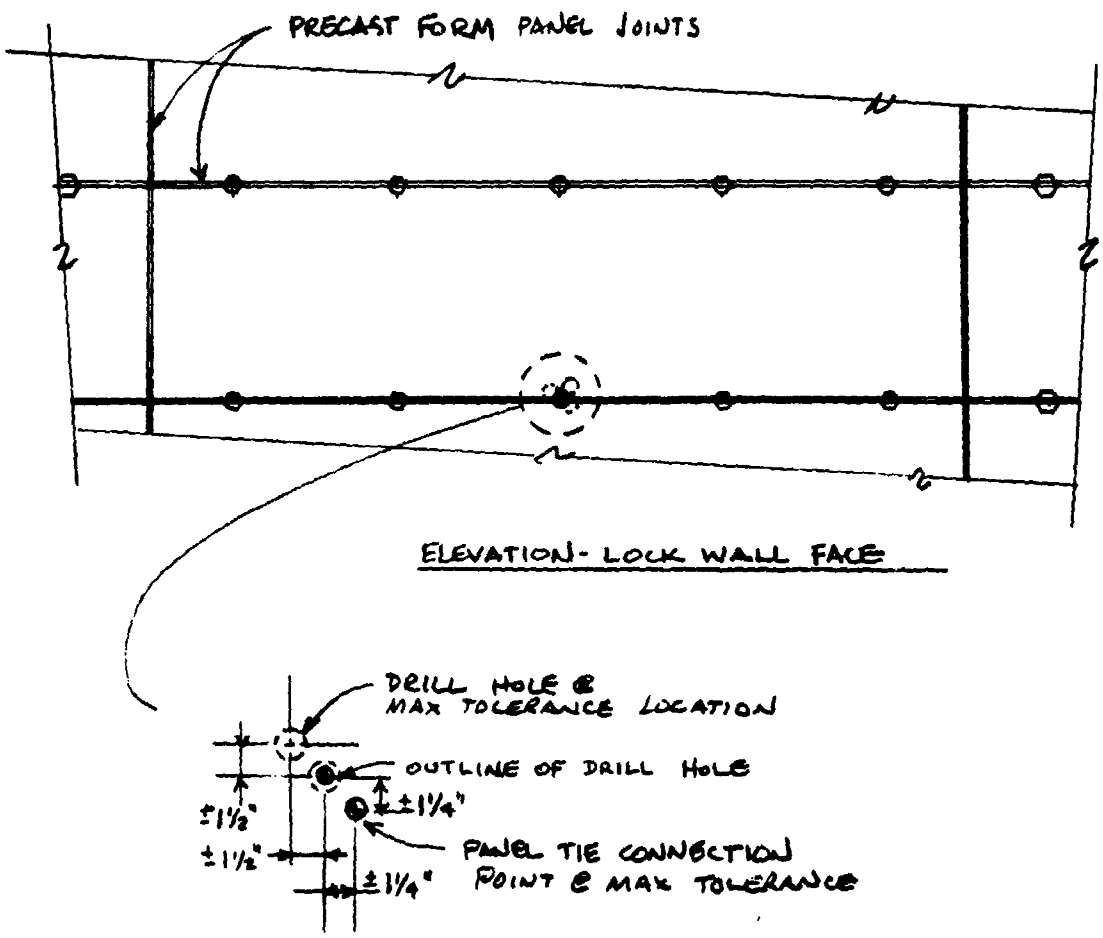
GEOMETRIC CONSTAINTS TIE SYSTEM



VERTICAL SECTION THRU WALL

ABAM
CONSULTING ENGINEERS

Sheet A11 of ______
Project COE LOCK REHABILITATION
STAY-IN-PLACE FORMS
Job No. AB6C9
Designer EWO
Subject TIE HARDWARE DETAILS
Date 11 APR 86



ELEVATION- LOCK WALL FACE

Sheet A12 of

ABAM CONSULTING ENGINEERS

Project COE LOCK REHABILITATION
STAY-IN-PLACE FORMS

Subject TIE · DESIGN

Job No. AB6029

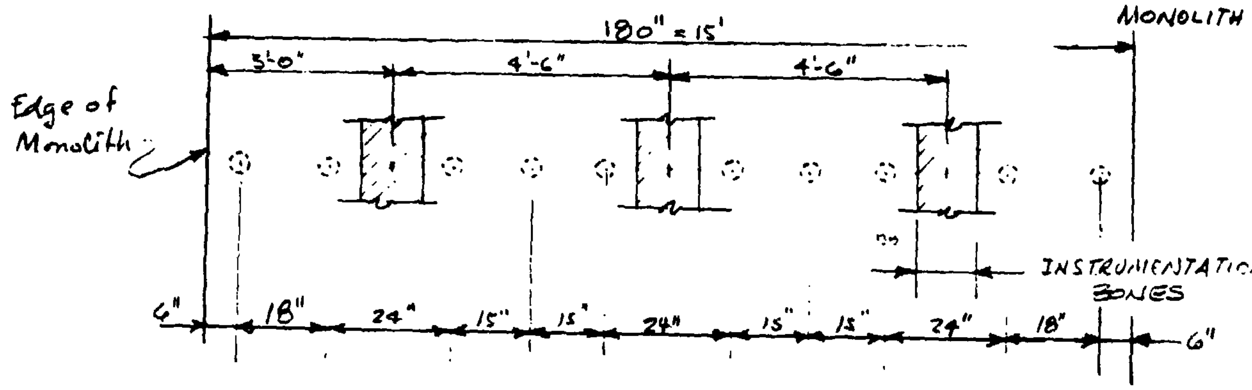
Designer EWO

Date 17 APR 86

TIE DESIGN METHODOLOGY (Test Monolith)

1. Exclude Ties from instrumentation zones as identified by COE
2. Keep tie spacing to approximately 2' maximum in order to obtain 1-way span action as assumed in panel design
3. Limit ℄ tie hole locations to no closer than 6" of monolith edge + instrumentation zone.
4. Tight tolerance control will be possible from relatively true test monolith faces.

RESULTANT TIE SPACING



TIE FORCES

Panels P-1, P-6, P-7, & P-8 will experience maximum tie load
Limiting pressure 1.25 ksf

$$\text{Maximum Tie Force} = 1.25 \times 2' \times \left(\tfrac{6}{2} + \tfrac{3}{2}\right) = 11.25^k$$

(4.5)

Average Tie Force accounts for all ties not being at 2' Spacing.

$$\text{Average Tie Force} = \frac{1.25 \times 15 \times 4.5}{10} = 8.4^k$$

Prototype (Maximum Tie Spacing Will Be Limited to 4' in order to maintain 1-way span on panels)

$$\text{Tie Force} = 1.25 \times 6 \times 4 \times 1.5 \approx 45^k$$

↑ tolerance adjustment

Ultimate Tie loads

Test Monolith $T_{u\,max} = 1.9 \times 11.25 = 21.4^k$

$T_{u\,ave} = 1.9 \times 8.4 = 16.0^k$

Prototype $T_{u\,max} = 1.9 \times 45^k = 86^k$

Sheet A13 of

Project COE LOCK REHABILITATION STAY-IN-PLACE FORMS

Job No. AB6027

Designer EAO

Subject TIE - DESIGN

Date 25 June 96

ABAM CONSULTING ENGINEERS

REBAR SIZES

$$Capacity = \phi f_y A_s = 0.9 \times 60 \times A_s = 54 A_s$$

Test Monolith

For T_{umax} $\quad A_s = \frac{21.4}{54} = 0.40 \text{ in}^2$

#6 $\quad A_s = 0.44 \text{ in}^2$

T_{uave} $\quad A_s = \frac{16}{54} = 0.30 \text{ in}^2$

#5 $\quad A_s = 0.31 \text{ in}^2$

Commercial Ties have higher factors of safety typically. For the test monolith, use #7 bars and consider measuring Tie forces so the concrete pressure can be more accurately quantified.

$$\text{Approx Factor of Safety} = \frac{0.6 \times 54}{11.25} = 2.9$$

Use #7 Bars for Test Monolith.

Prototype

$$A_s = \frac{86}{54} = 1.58 \text{ in}^2$$

Appears that prototype tie spacing will have to be limited to approximately 2' in order to limit bar size to #8.

$$T = 1.25 \times 6 \times 2 \times 1.5 = 22.50^k$$

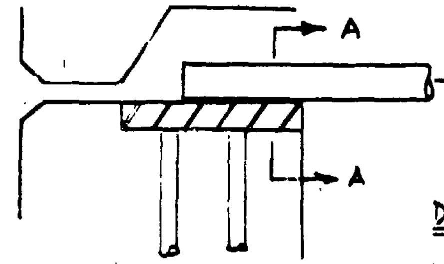
$$T_u = 1.9 \times 22.50 = 42.75''$$

$$A_s = \frac{42.75}{54} = 0.79 \text{ in}^2$$

#8 Bar $\quad A_s = 0.79 \text{ in}^2$

ABAM CONSULTING ENGINEERS	Sheet A14 of ____
Project COE LOCK REHABILITATION STAY-IN-PLACE FORMS	Job No. A86029
	Designer EWO
Subject TIE WELD PLATE DESIGN	Date 25 June

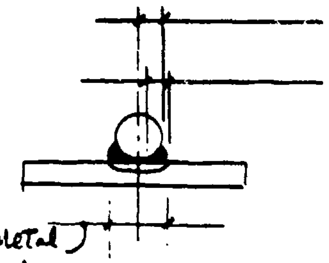
DESIGN CONNECTION TO TEST MONOLITH PANELS



$T_u = 11.25 \times 2.90 = 32.63\ k$
F of S

DESIGN WELD

Use Flare-bevel-groove weld



s = Weld Size = Rebar Radius = 7/16"

E = Effective Throat = 0.4 s = 0.175 in

use E70XX

Base Metal Shear Area

$T_u = \phi \times 42 \times A_w$

$A_w = \frac{32.63}{0.9 \times 42} = 0.863$

$A_w = l \times E$ (l includes both welds)

$l = \frac{0.863}{0.175} = 4.93"$

Length of Each Weld 2 1/2"

Check Base Metal Shear stress

$F_v = 0.4 \times 36 = 14.40$ ksi

$A_w = \frac{11.25 \times 29/1.9}{144} = 1.192\ in^2$

$l = \frac{1.192}{0.875} = 1.36\ in^2$

$l = 3/4"$ each leg. OK

Quick Check - Prototype Welds.

Assuming pressures are better defined & lower factors of safety are justified

Assume higher strength weld & base metals are used

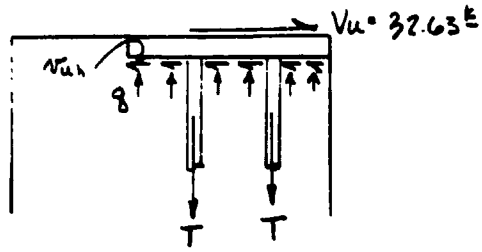
Electrodes E80XX $A_w = \frac{42.75}{48} = 0.891\ in^2$ $l = \frac{0.891}{210.2} = 2 1/4$

Base Metal $F_v = 0.4 \times 42 = 16.8$ ksi

$l = \frac{22.50}{16.8 \times 1"} = 1.34$ in

Project COE LOCK REHABILITATION STAY-IN-PLACE FORMS	Job No. A96029
	Designer EWO
Subject WELD PLATE DESIGN	Date 27 June 96

ABAM CONSULTING ENGINEERS



$V_u = v_{uh} \times A_c$

v_{uh} = shear stress

A_c = contact Area

$v_{uh} = q\,\mu$

q = bearing stress

μ = coefficient of friction = 0.7

$q = \frac{T}{A_c}$

$\therefore V_u = \phi \frac{T}{A_c} \mu A_c = \phi T \mu$

$T_{reqd} = \frac{32.63}{0.85 \times 0.70} = 54.84^k$

Use Deformed Bar Anchors to Embed Plate

$T = \phi A_b F_y = 0.9 \times A_b \times 60 = 54 A_b$

Bar Size	A_b	T	No of Bars Reqd
3/8	0.11	5.94^k	9.23
4/8	0.20	10.8^k	5.08
5/8	0.31	16.7^k	3.3

<u>Use 6 - #4 Deformed Bar Anchors.</u> $T_{act} = \frac{54.84}{6} = 9.14^k$

$V_u / bar = \frac{32.63}{6} = 5.44\ ^k/bar$

Capacity $\phi V_s = 0.75 A_b f_s = 0.75 \times 0.20 \times 60 = 9.0^k$

<u>Check Combined Stresses.</u>

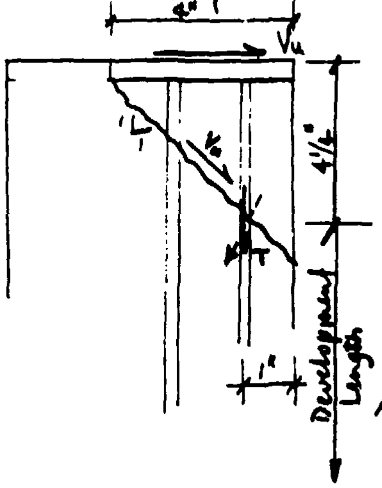
$\left(\frac{T_u}{\phi T_{us}}\right)^2 + \left(\frac{V_u}{\phi V_{us}}\right)^2 \leq 1 \quad \left(\frac{9.14}{10.8}\right)^2 + \left(\frac{5.44}{9.0}\right)^2 = 0.716 + 0.365$

$= 1.08 > 1.0$

This is slightly over stressed but considering that an additional factor of safety above normal Load Factor is being used, the Connection should be OK

ABAM CONSULTING ENGINEERS	Sheet A16 of ___
Project COE LOCK REHABILITATION	Job No. A86029
STAY-IN-PLACE FORMS	Designer EWD
Subject WELD PLATE DESIGN	Date 27 June 86

Assuming Diagonal Crack Occurs, verify if adequate reinforcement is provided across crack.



$V_u = 32.63 \times \sqrt{2} = 46.15^k$

Check Concrete Capacity

$\phi V_c = \phi\, 2\sqrt{f_c'}\, A_c$

$A_c = 4\sqrt{2} \times 5 + 2(\frac{1}{2} \times 4 \times 4) = 44.28\ in^2$

$\phi V_c = \dfrac{0.85 \times 2\sqrt{6500} \times 44.28}{1000} = 6.07^k \ll 32.$

Must Provide Shear Friction Reinf.

$V_u = \phi A_{vf} f_y \mu$

$A_{vf} = \dfrac{46.15}{0.85 \times 60 \times 1.4} = 0.646\ in^2$

6 - #4 Bars Provide $A_s = 6 \times 0.20 = 1.20\ in^2$

<u>Check Development Length</u>

$l_d = \dfrac{0.04 \times 0.20 \times 60000}{\sqrt{6500}} = 5.95\ in$

or

$l_d = 0.0004 \times 0.5 \times 60000 = 12.0\ in$

Req'd Bar Length $= 12 + 4\frac{1}{4} = 16\frac{1}{4}$"

#4 - Deformed Bar Anchor x 18" LONG

Sheet A17 of

ABAM CONSULTING ENGINEERS

Project: COE LOCK REHABILITATION — STAY-IN-PLACE FORMS

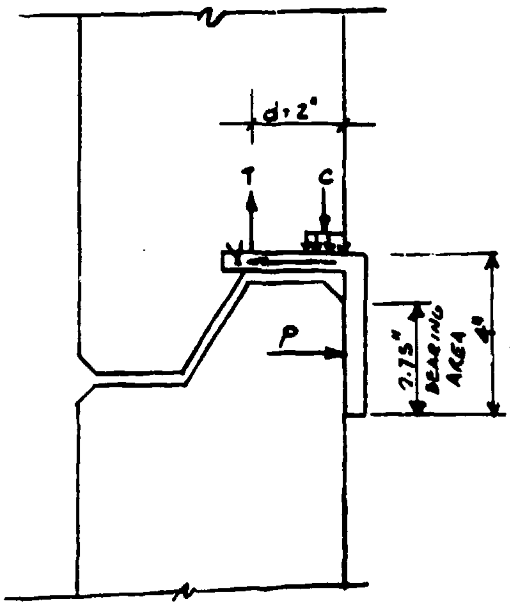
Job No. A86029

Designer: EWO

Subject: ALIGNMENT ANGLE DESIGN

Date: 24 June 86

ALIGNMENT ANGLE DESIGN



DESIGN FORCE

P = concrete pressure over 1/2 panel width

$\therefore$ $P = 1/2 \times 1.25\ ksf \times 6' max = 3.75\ k/ft$

$P_{ULT} = 1.9 \times 3.75 = 7.13\ k/ft$

$F_{BRG} = \phi(0.85 f_c' A_1)$

$= 0.70 \times 0.85 \times 6.5 \times 2.75 \times 12"$

$F_{BRG} = 128^k >> 7.13^k$ ok.

Check Angle Size

$M = 3.75 \times (4 - \frac{2.75}{2}) = 3.75 \times 2.63 = 9.84\ k\text{-}in/ft$

$F_b = 0.75 f_y = .75 \times 36 = 27\ ksi$

$S_{req'd} = \frac{9.84}{27} = 0.365\ in^3$

$S = \frac{bt^2}{6} \quad t^2 = \frac{6S}{b} = \frac{6S}{12}$

$t_{req'd} = \sqrt{\frac{0.365}{2}} = 0.43\ in^3$

Check Shear

$F_v = 0.4 \times F_y = 0.4 \times 36 = 14.4\ ksi$

$f_b = \frac{7.13}{0.5 \times 12} = 1.19\ ksi < 14.4\ ksi$ ok.

Check Bending in Embedded Leg of ∠

From previous calcs $k = 0.4191$ $\quad n = 5.92$

$j = 0.8603$ $\quad f_c = 0.45 \times 6500 = 2925\ psi$

$R = 527.3\ psi$

$bd^2 = \frac{M}{R}$ $\quad M = bd^2 \times R = 12 \times 2^2 \times 0.527$

$M = 25.30\ k\text{-}in/ft >> 9.84\ k\text{-}in/ft$ ok

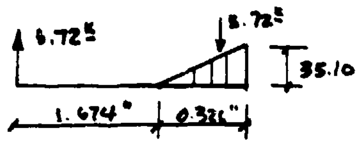
ABAM CONSULTING ENGINEERS	Sheet A18 of
Project: COE LOCK REHABILITATION / STAY-IN-PLACE FORMS	Job No. A86029
Subject:	Designer: EWD
	Date: 24 June 96

$M = Tjd$

$T = \frac{9.84}{0.8603 \times 2} = 5.72^{k}$

$T = C = \frac{1}{2} f_c b x$

$x = \frac{2 \times 5.72}{2.923 \times 2} = 0.326$



LOADING DIAGRAM

SHEAR

$M = 5.72 \times 2 + 5.72 \times \frac{0.326}{3} = 12.06$ k·in/ft

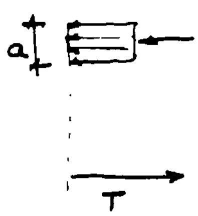
For true equilibrium, M should be = 9.84 k·in. The moment is higher due to approximate analysis (ie $f_c < 2925$ ksi). This approach will lead to conservative design.

$S_{req'd} = \frac{12.06}{27} = 0.447 \text{ in}^3$

$t = \sqrt{\frac{0.447}{2}} = 0.47$ in

½" L OK

Stud will be provided to develop T. However, design is based on ultimate $M_u = 7.13 \times (4 - \frac{2.75}{2}) = 18.72$ k·in



$M = T(d - \frac{a}{2})$ $\quad T = C = 0.85 f'_c b a$

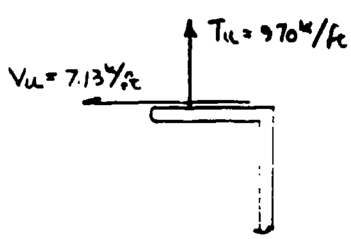
Select T so that a is equal & C & T are in equilibrium.

$T = 9.70^{k}$ $\quad a = (2 - \frac{1.9 \times 9.84}{9.70}) \times 2 = 0.145$ in

$C = 9.70$ $\quad a = \frac{9.70}{0.85 \times 6.5 \times 12} = 0.146$ in OK

$\therefore M_u = 9.70(2 - \frac{0.146}{2}) = 18.7$ k·in ≈ 18.72 k·in OK.

DESIGN OF ANGLE CONNECTION TO PRECAST PANEL



Weld rebar to top of angle to resist Vu

#3 $\quad V_u = 0.11 \times 60 \times 0.9 = 5.94$ k/bar
spacing = 10" oc

#4 $\quad V_u = 0.20 \times 60 \times 0.9 = 10.8$ k/bar
spacing = 18" oc

Project: COE LOCK REHABILITATION	Sheet: A19 of
STAY-IN-PLACE FORMS	Job No. A96079
Subject:	Designer: EnJO
	Date: 25 June 86

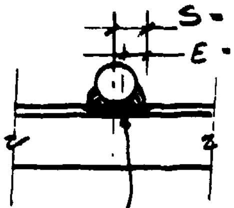
ABAM CONSULTING ENGINEERS

Check Welding Requirements

Use #3 deformed bar anchors

Use Flare Bevel Groove Welds

Show 1/4" weld on drawings to heat up base metal & better fusion.



S = Weld Size = Rebar Radius = $1/2 \times 3/8 = 3/16$"

E = Effective Throat = $0.4 \times S = 0.4 \times 3/16 = 0.075$ in

Assume E70xx Weld Metal

$V_u = \phi\, 42 \times A_w \times 2$ welds per bar.

$A_w = \dfrac{V_u}{2\phi \times 42} = \dfrac{7.13}{2 \times 0.85 \times 42} = 0.100\ in^2$

$A_w = E \times l$

$l = \dfrac{0.100}{0.075} = 1.33$ in

Check Stresses of Base Metal

$F_v = 0.4 \times 36 = 14.40$ ksi

$f_v = \dfrac{V}{A_w}$ $\quad A_w = \dfrac{7.13}{1.9 \times 14.4 \times 2} = 0.130\ in^2$

$l = \dfrac{0.130}{0.375} = 3/8$ in.

Weld Specification $\dfrac{3/16\ (0.075)}{3/16\ (0.075)}$ 1 1/2" / 1 1/2"

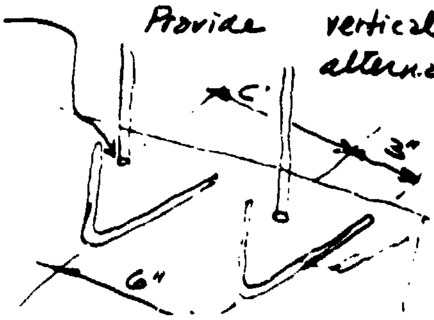
Tension Stud

$T_u = 9.70$ k/ft

Spacing $= \dfrac{5.94}{9.70} \times 12 = 7.3$" oc.

Provide vertical & horizontal anchors at alternating 6" centers

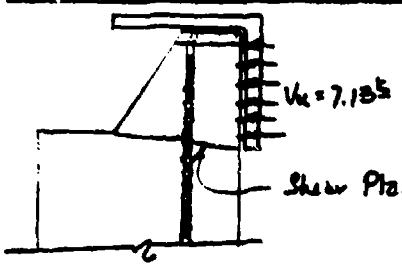
Use Automatic Stud Welding Equipment for straight bar welds



ABAM
CONSULTING ENGINEERS

Project: COE LOCK REHABILITATION — STAY-IN-PLACE FORMS
Subject:
Sheet: A20 of
Job No. A86029
Designer: END
Date: 25 June 96

Check Shear Stress on Lower Panel Key



Concrete Capacity:

$V_{uc} = \phi\, 2\sqrt{f'_c}\, b_w d$

$V_{uc} = \dfrac{0.85 \times 2\sqrt{6500} \times 12 \times 4''}{1000} = 6.58^k < 7.13$

Assume Crack Occurs & Entire Load Must Be Carried In Shear Friction.

$V_u = \phi A_{vf} f_y \mu$

$A_{vf} = \dfrac{7.13}{0.85 \times 60 \times 1.4} = 0.100 \text{ in}^2/\text{ft}$

#3 @ 12" oc nominal

Weld #3 deformed bar anchors to base plate. Check minimum size plate req'd

$f_{BRG} = 0.70 \times 0.85 \times 6.5 \times A \qquad A = 2.5'' \times \ell$

$\ell = \dfrac{7.13}{0.70 \times 0.85 \times 6.5 \times 2.5} = 0.74''$

Use 1/2" x 1" x 2½ minimum anchor plate

Check Need for Shear-Friction Reinforcement for Test Monolith

Maximum shear will occur only for 6' wide panels. (Only panels P-1 & P-7) Panel P-1 is located at bottom of wall and has form ties at top & bottom of the panel. Therefore panel P-7 will be only panel which may require additional shear-friction steel. Verify if panel will be subjected to 1.25 ksf pressure.

ABAM CONSULTING ENGINEERS	Sheet A21 of ___
Project COE LOCK REHABILITATION STAY-IN-PLACE FORMS	Job No. A86023
Subject ALIGNMENT ANGLE DESIGN	Designer [illegible]
	Date 26 June 86

CONCRETE PRESSURE: Determine per recommendations of ACI Committee 347

TABLE 5-2: MAXIMUM LATERAL PRESSURE FOR DESIGN OF WALL FORMS

Based on ACI Committee 347 pressure formulas

NOTE: Do not use design pressures in excess of 2000 psf or 150 × height of fresh concrete in forms, whichever is less

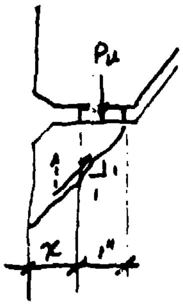
Rate of placement, R, ft per hr	p, maximum lateral pressure, psf, for temperature indicated					
	90F	80F	70F	60F	50F	40F
1	250	262	278	300	330	375
2	350	375	407	450	510	600
3	450	488	536	600	690	[illegible]
4	550	600	664	750	870	[illegible]
5	650	712	793	900	1050	[illegible]
6	750	825	921	1050	1230	[illegible]
7	850	938	1050	1200	1410	[illegible]
8	881	973	1090	1246	1466	1795
9	912	1008	1130	1293	1522	1865
10	943	1043	1170	1340	1578	1935

With concrete temperatures in the 50 to 60 degree range, low placement rates would be required to remain below 1.25 ksf pressure.

Therefore, provide deformed bar anchors across potential crack plane. Provide a minimum of 15 anchors in the keyway of panel P-6 which would resist the load of Panel P-7.

<u>Check Also Bearing Stresses Between Panels</u>

Load on Neoprene See calcs titled - Joint Details - Neoprene Sec.



$P = \text{Area} \times \text{Stress} \approx \dfrac{12\,in^2 \times \left(\frac{8}{28} \times 717\right)}{1000} = 2.5\ ^k/ft.$

$P_u = 1.5 \times 2.5 = 3.75\ k$

Approx Shear Plane $\ell = 1.5 \times \sqrt{2} = 2.12$ in

$V_{uc} = \dfrac{0.85 \times 2 \times \sqrt{6500} \times 2.12 \times 12}{\sqrt{2}} = 2.47^k$

Move neoprene further into panel to increase shear plane

$x = 1\frac{1}{2}"$ $V_u = \dfrac{0.85 \times 2 \times \sqrt{6500} \times 2.5 \times 12 \times \sqrt{2}}{\sqrt{2} \times 1000} = 4.11^k$ OK

ABAM
CONSULTING ENGINEERS

Project COE LOCK REHABILITATION
STAY-IN-PLACE FORMS
Subject JOINT DETAILS
NEOPRENE SEALS

Job No. [illegible]
Designer [illegible]
Date 29 APR 96

PANEL WEIGHT

Min - 3.3 k/ft.
Max - 8.6 k/ft

Seals will be compressed until contact is made w/ shims which will ensure required gap is maintained. Therefore, select neoprene dimension & durometer which will compress seals to required thickness

Although neoprene should be limited to 15% strain under compression, this criteria is imposed to limit long term creep rate and long term weathering. Based on 15% limit, neoprene thickness would be limited to:

$$t = \frac{0.25''}{0.85} = 0.29 \text{ in}$$

Use 5/16" seal - 50 durometer (softest avail.)

1" wide seal $\quad S_f = \frac{1 \times 12}{(12+12) \times 5/16} = 1.6$

$$f_c = \frac{3300}{1 \times 12} = 275 \text{ psi} \quad \text{or} \quad \frac{8600}{12} = 717 \text{ psi}$$

% Strain = 14% $\qquad$ % Strain = 28%

1½" wide seal $\quad S_f = \frac{1.25 \times 12}{(12+12) \times 5/16} = 2.0$

$$f_c = \frac{3300}{15} = 220 \text{ psi} \quad \text{or} \quad f_c = \frac{8600}{15} = 573 \text{ psi}$$

% Strain = 9% $\qquad$ % Strain = 21%

Deflected Ht = 0.91 × 5/16 = 0.28 in $\qquad$ % Strain = 17%

1½" wide seal will not compress enough. Therefore, use 1" wide seal. Maximum Actual Strain will equal

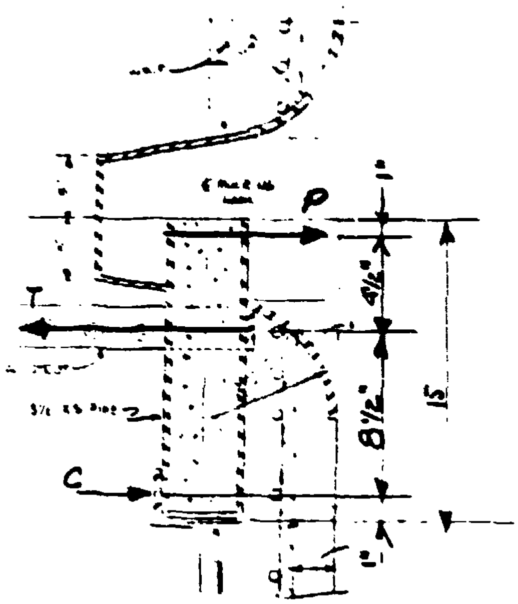
$$\% = \frac{1/16}{5/16} = 20\%$$

This should be acceptable since creep is prevented by shims

LINE HOOK DESIGN

Use A-36 Steel as a minimum - better if req'd.

Line Hook Load: Use 1/2 full Scale Load $P = \frac{130}{2} = 65^k$



$\Sigma M_{oc} = 0$

$T = \frac{13 \times 65}{8.5} = 99.4^k$

$C = 99.4 - 65 = 34.4^k$

Shear

Moment

$M_{max} = 99.4 \times 4.5 = 447.4$ K-in

Req'd Section Modulus

$S = \frac{447.4}{0.66 \times 36} = 18.83 \text{ in}^3$

6" Double X-Strong $S = 20 \text{ in}^3$

PRELIMINARY GEOMETRY

Solid Section

$S_{req'd} = \frac{447.4}{0.75 \times 36} = 16.57 \text{ in}^3$

$S = \frac{\pi d^3}{32} = 16.57 \text{ in}^3$

$d = 5\frac{1}{2}"$

Reduce Working Load to 50 kips Use Steel w/ $F_y = 50$ ksi

$T = 76.5$ k $M = 344$ kin

$S_{req'd} = 344/.75 \times 50 = 9.18 \text{ in}^3$

$d_{req'd} = \sqrt[3]{\frac{9.18 \times 32}{\pi}} = 4\frac{1}{2}"$

Project COE LOCK REHABILITATION — STAY-IN-PLACE FORMS
Job No. A96229
Designer EWJ
Subject LINE HOOK DESIGN
Date 25 APRIL 96

ABAM CONSULTING ENGINEERS

Check Shear Stress

$A = \frac{\pi \times 4.5^2}{4} = 15.9\ in^2$

$f_v = \frac{65}{15.9} = 4.09\ ksi$

$F_v = 0.4 F_y = 0.4 \times 50 = 20\ ksi$ OK

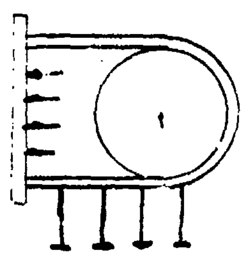
Drag Strut Design

$T = 76.5^k$ Force/Leg $= \frac{76.5}{2} = 38.25^k$

$A_{req'd} = \frac{38.25}{0.6 \times 36} = 1.77\ in^2$

Assuming 2" PL $t = 0.88"$
3" PL $t = 0.59$ Use 5/8" x 3" strap

Check Brg Stresses $A = 3 \times \pi \times \frac{4.5}{2} = 21.2\ in^2$



$f_{brg} = \frac{76.5}{21.2} = 3.61\ ksi$

$F_{brg} = 0.9 \times 36 = 32.4\ ksi$ OK

Determine No of Studs to Transfer Load.

1/2"ϕ Stud $V = 8.83^k$

No req'd $= \frac{38.25}{8.83} = 4.33$ Studs

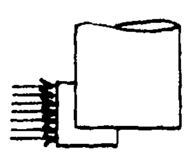
Min Cold Bending Radius per AISC Table on Pg 4-166 is $2t = 2 \times 5/8 = 1\frac{1}{4}"$

Use 4 Studs on each Leg of Strap-
Brg plate will provide add'l redundancy

Kicker Angle Design

$C = 76.5 - 50 = 26.5^k$

$C_u = 1.7 \times 26.5 = 45^k$



$F_{brg} = 0.7 \times 0.85 \times 4000 \times A$

$A_{req'd} = \frac{45}{2.38} = 18.93\ in^2$

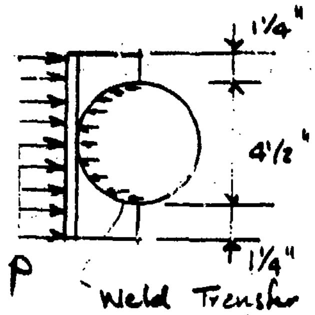
Try 3" ∠ x 7" Long.

Sheet A25 of

Project COE LOCK REHABILITATION
STAY-IN-PLACE FORMS
Subject LINE HOOK DESIGN

Job No A36029
Designer Edo
Date 25 APRIL 86

ABAM CONSULTING ENGINEERS



$p = \frac{26.5}{7} = 3.8$ k/in

$M = \frac{3.8 \times 1.25^2}{2} = 2.97$ k-in

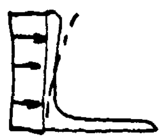
Check $\frac{3.8 \times 3.5^2}{2} = 23.3$ k-in

Design $M = 15$ k-in

$S_{req'd} = \frac{15}{0.6 \times 36} = 0.69 \text{ in}^3$

∠ 3×3×3/8 Would Work.

Check Vertical Bending



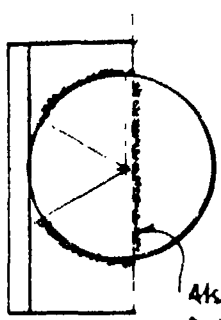
$p = \frac{3.8}{3} = 1.27$

$M = \frac{1.27 \times 2.625^2}{2} = 4.36$ k-in

$S_{req'd} = \frac{4.36}{0.75 \times 36} = 0.16 = \frac{bt^2}{6}$

$t = 1.0"$

Maximum Angle Size Available ½". The above load distribution assumes 1-way distribution. However considerable load will be transfered horizontally due to nearness of pin & ½" ∠ should work. Note also that we've assumed all C load into angle. In actuality, transfer will be along full length of pin & thus load on angle will be less.



Also weld available here ≈ 4"

Size Weld - Assume 2-60° Sectors are accessible

$L = \frac{120}{360} \times 2 \times \pi \times 2.25 + 4 = 8.71"$

Force/inch of Weld $= \frac{26.5}{8.71} = 3.04$ K/in

Force/16 $= 0.3 \times 70 \times 0.707 \times 1/16 = 0.928$ k/in

Weld Size $= \frac{3.04}{0.928} = 3.28$

Use 4/16" weld (1/4")

Note Use 5/16" weld for more heat & better fusion

Project COE LOCK REHABILITATION STAY-IN-PLACE FORMS
Job No. A86029
Designer EWD
ABAM CONSULTING ENGINEERS
Subject MOORING HOOK
Date 28 APRIL 96

Determine No of Dowels to Transfer Load Into Lock Wall

Use #6 Bars $F/bar = 0.44 \times 24 = 10.56^k$ Working Strength

No of Bars $= \frac{50}{10.56} = 4.73$ bars.

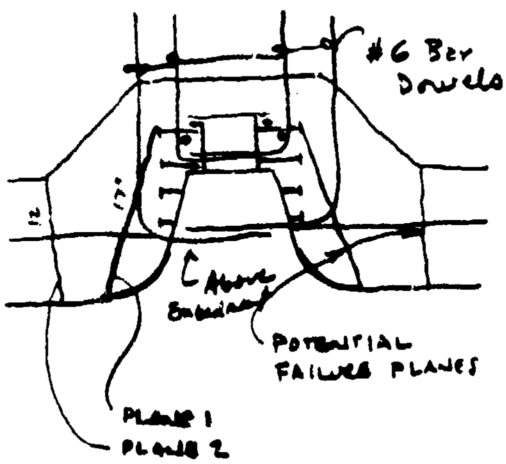
OR

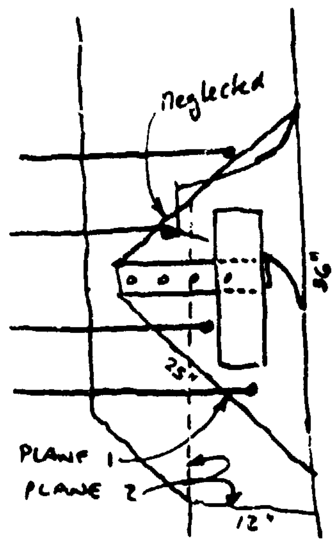
$Fu/bar = 0.44 \times 60 \times 0.90 = 23.76$

No of Bars $= \frac{1.9 \times 50}{23.76} = 4.0$ bars

Provide 3 pairs minimum – 6 Total.

Check Shear Stresses on Pullout Surface





Shear Plane 1

Area ≈ $2 \times 25 \times 36 + 25 \times 24 = 2400\ in^2$

$v_u = \frac{1.7 \times 50\,000}{2400} = 35.4$ psi

$v_{cu} = 0.85 \times 2\sqrt{4000} = 107.5$ psi

↑ neglects contribution of precast form panel @ $f'_c = 6500$ psi

Sheet A27 of ____

Project COE LOCK REHABILITATION Job No. A80029

STAY-IN-PLACE FORMS Designer EWJ

Subject LINE HOOK DESIGN Date 29 APRIL 96

ABAM CONSULTING ENGINEERS

Check Shear Plane #2

$A_{z} = 36 \times 12 + 2 \times (12 \times 30 + \frac{1}{2} \times 12 \times 12) = 1296 \text{ in}^2$

Bottom Sides Top Plane Neglected

$\bar{v}_u = \frac{1.7 \times 50{,}000}{1296} = 65.6 \text{ psi}$

Although calculations indicate no reinforcement is required across shear planes, provide reinforcement to assure redundancy.

$V_s = \phi A_{vf} f_y \mu \sin\alpha$

← assume all steel @ 45° w/r to shear plane.

$A_{vf_{req'd}} = \frac{17 \times 50}{0.85 \times 60 \times 1.4 \times \sin 45°} = 168 \text{ in}^2$

No of #6 Bars $= \frac{1.68}{0.44} = 4$ bars minimum

Sheet A28 of

Project COE LOCK REHABILITATION
STAY-IN-PLACE FORMS

Subject HARDWARE DESIGN

Job No. 396029

Designer EWO

Date 23 APR 96

ABAM
CONSULTING ENGINEERS

VERTICAL ALIGNMENT HARDWARE

Compute Load on Alignment Screws

Eliminated 5 May 96 → 3' PANEL WEIGHT (5") $5/12 \times 0.160 \times 3 \times 15' \times 1.10 = 3.3^k$

10% Allowance for thickenings & hardware

3' PANEL WEIGHT (6½") $6.5/12 \times 0.160 \times 3 \times 15' \times 1.10 = 4.3^k$

6' Panel Weight (6½") $2 \times 4.3 = 8.6^k$

WEIGHT PANELS P-1 TO P-5 $W = 8.6 + 3 \times 3.3 + 4.3 = 22.8^k$

PANELS P-5 TO P-10 $W = 4.3 + 2 \times 3.3 + 8.6 + 4.3 = 23.8^k$

5/5/96 Rev'd $W = 6.5/12 \times 15 \times 20 \times 0.160 = 26.0^k$

NOTE: This does not include the weight of fresh concrete which will act on panels. Maximum possible weight that is available.

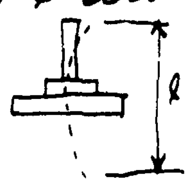
$W_{max} = 12/12 \times 0.160 \times 20 \times 15 = 48^k$

½ to Panel

Use Min of 4 Screws $R = \frac{24+26}{4} = 12.5^k$

Size Alignment Screw Use ASTM A-307

Try 1"φ Bolt $r = \frac{d}{4} = \frac{1.0}{4} = 0.25$ in



$\frac{k\ell}{r} = \frac{2.0 \times 4.75}{0.25} = 38$

$F_a = 19.35$ ksi AISC Table 3-36

$f_a = \frac{12.5}{0.7854} = 15.91$ ksi OK

Check Concrete Stresses/Size Base Plates

$P_u = 1.9 \times 12.5 = 23.75$ kips

$F_{Bec} = \phi(0.85 f_c' A_1)$

$A_{Req'd} = \frac{23.75}{0.7 \times 0.85 \times 4} = 8.57\ in^2$ (Monolith)

4" ϕ PL OK.

$A_{Req'd} = \frac{20.4}{0.7 \times 0.85 \times 6.5} = 6.14\ in^2$ (P/C Panel)

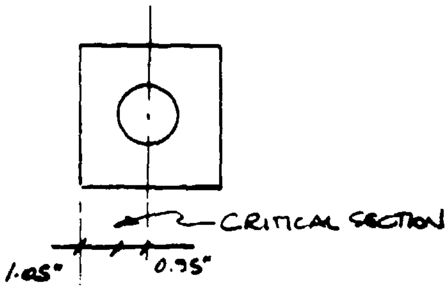
4" ϕ PL Provides $16 - \frac{\pi \times 1.9^2}{4} = 13.16\ in^2$

℄ less pipe

OK Use 3/4" Thick Base Plates

Check Base Plate Bending

Top Plate $p = \frac{12.5}{13.16} = 0.95$ ksi



$M = \frac{0.91 \times 1.06^2}{2} = 0.52$ k·in/in

$= 2.09$ k·in

$S = \frac{4 \times 1/2^2}{6} = 0.17\ in^3$

$f = \frac{2.09}{0.17} = 12.32$ ksi

$F_b = 0.75 \times 36 = 27$ KSI OK.

Lower Plate $p = \frac{12}{16} = 0.75$ ksi

Critical Section is at face of bolt head

$x = 1^7/8/2 = 0.9375$ in

$M = \frac{0.75 \times (2 - 0.9375)^2}{2} \times 4 = 1.69$ ksi

Top Plate Governs.

APPENDIX B: CONSTRUCTION SPECIFICATIONS PHASE II DEMONSTRATION PROJECT

SECTION 1
GENERAL PROVISIONS

1.1 GENERAL

1.1.1 Description of Work

a. This work consists of demonstrating the feasibility of repairing deterioriated navigation lock wall surfaces through the use of precast, stay-in-place concrete form panels in conjunction with cast-in-place concrete bonding layers. This work will be performed on dry land at the Corps of Engineers, Waterways Experiment Station, using an existing one-half scale lock wall mock-up section.

b. The work includes providing all materials, labor, inspections, tests, and supervision required for a complete installation as shown on the drawings and described in these specifications.

1.1.2 Definitions

a. Corps: Shall mean the Corps of Engineers who is responsible for commissioning the work. The Corps shall have the final determination regarding interpretation of contract drawings and specifications.

b. Engineer: Shall mean ABAM Engineers Inc. who is responsible for design of the precast concrete stay-in-place forming system and who shall perform reviews of Contractor's submittals, independent inspections, and surveillance of the work, and whose personnel will be available to provide direction of the work and interpretation of the design requirements.

c. Contractor: Shall mean Premier Waterproofing, Inc. who is responsible for performing all work as described in the specifications and on the drawings.

1.1.3 Prosecution of the Work

a. The stay-in-place precast form demonstration project will be carried out at the Corps' Waterways Experiment Station in Vicksburg, MS. Contractor shall be required to abide by Corps' rules and regulations regarding use and access to Corps' facilities. Such regulations shall include requirements for environmental protection, health and safety, security and labor relations.

b. Corps' and Engineer's personnel will be performing inspections and tests in conjunction with Contractor's work. Full cooperation shall be given to Corps' and Engineer's personnel to install instrumentation, monitor and inspect the work.

c. Direction of the work will be given by the Engineer. In case of conflict, Contractor shall notify Engineer for resolution.

1.1.4 Quality Control

a. Contractor shall perform all work in accordance with referenced specifications and standards.

b. Contractor shall maintain records of tests and inspections as required to demonstrate compliance with referenced standards. Such records shall be made available to the Engineer or to the Corps upon request.

1.1.5 Submittals

a. Contractor shall submit copies of all required drawings, test results, inspections, and documentation as required by these specifications to the Engineer. Contractor shall allow 7 days for review of submittals by Engineer.

b. Submittals will be reviewed by the Engineer for compliance with the contract requirements and returned either reviewed without comment, disapproved, or reviewed with comment. For those submittals disapproved or reviewed with comments, Contractor shall make the required changes and resubmit such submittals for rereview.

c. Contractor shall not proceed with work contingent upon Engineer's review until satisfactory review of necessary submittals has been completed, unless written direction to proceed is received from the Engineer.

d. Contractor shall supply samples of materials used in the work to Corps' or Engineer's personnel as requested. Such samples will be considered incidental to the work and will be used by the Corps or Engineer to verify material properties or performance.

1.1.6 Housekeeping and Cleanup

a. Contractor shall maintain work area in a neat and orderly fashion. Work area shall be cleaned daily at the close of work.

b. Refuse, shipping and packaging materials, wasted material and products, and discarded samples shall be disposed of as required by the Corps.

1.1.7 Project Photographs

The Contractor shall submit five copies of a photographic report consisting of approximately 30 photographs showing the different stages of construction. Photographs shall be taken by a competent photographer and shall be black and white, standard commercial quality, 8 x 10 in. in size, and on single-weight glossy paper. The negatives of all photographs shall also be submitted.

The photographs shall be enclosed in standard three-ring binders in back-to-back double-faced plastic sleeves. Each print shall have an information band along the front bottom edge with a description of the photograph's content, reference negative number, and date photograph was taken.

As a minimum, pictures of the following items or activities shall be included in the report:

- Formwork and formwork details
- Reinforcement installed in the forms
- Steel hardware with views of hardware positioned in the forms
- Precast panel casting operation
- Panel handling and transportation
- View of the existing monolith with surface prep completed
- Tie and dowel hole drilling
- Tie and dowel installation
- Tie and dowel testing
- Panel installation including completed tie connection, installation of seals and grout layer.
- Placement of the infill concrete
- Details of auxiliary formwork and reinforcement for the closure cap
- General view of the completed installation

SECTION 2
PRECAST CONCRETE

2.1 GENERAL

2.1.1 Work Included

The work includes all materials and workmanship required for fabrication, delivery, handling, and erection of precast concrete leave-in-place form panels.

2.1.2 Related Work

Related work includes fabrication of lock hardware and appurtenances and production and placing of cast-in-place concrete work.

2.1.3 References

a. ACI 301: Structural Concrete for Buildings

b. ACI 318: Building Code Requirements for Reinforced Concrete

c. PCI Design Handbook: Precast and Prestressed Concrete

d. PCI MNL-116: Manual for Quality Control for Plants and Production of Precast Prestressed Concrete Products

2.1.4 Submittals

a. Contractor shall submit shop and erection drawings for all precast elements. Drawings shall indicate fabrication details, reinforcing, connection details, support items, dimensions, and temporary attachments and work.

b. Contractor shall submit details showing proposed methods of lifting, handling, storing, and erecting precast elements.

c. Weights of precast elements shall be computed and listed on shop drawings.

d. Contractor shall submit results of tests on materials as specified in Paragraphs 2.2.1 and 2.3.3.

e. Concrete mixture designs and qualifying data.

f. Catalog cuts of other miscellaneous products to be incorporated into the work such as nonshrink grout, joint seals, tie and dowel bonding agents, etc.

2.1.5 Quality Control

a. All work shall be performed in accordance with PCI Design Handbook - Precast and Prestressed Concrete, and ACI 301 Structural Concrete for Buildings.

b. Precast concrete work shall be performed by experienced fabricators qualified in accordance with PCI MNL-116, Manual for Quality Control for Plants and Production of Precast Prestressed Concrete Products.

2.1.6 Delivery, Storage, and Handling

a. Delivery, storage, and handling of precast concrete elements shall be performed in such a manner as not to adversely affect their appearance or use. Panels shall not be lifted from the forms until panel strength has reacn 0.7 f'_c.

b. Design of lifting embedments and handling devices shall be the responsibility of the Contractor. Contractor shall provide details of proposed lifting methods, attachments, and devices for engineer's review. Handling embeds shall not be installed in the exposed exterior face of the panels.

c. Panels shall be lifted only from suitably designed lifting hardware embedded into the panel or with the use of slings properly placed and rigged to prevent damage to the panels.

d. Panels shall be adequately supporteo at all times with suitable cribbing and bracing during shipping and storage to prevent inadvertent damage from incidental loads or movements.

2.2 PRODUCTS

2.2.1 Materials

a. Concrete: Precast concrete materials shall conform to the following requirements:

- o Cement: Cement shall conform to the requirements of ASTM C 150.
- o Aggregate: Aggregate shall conform to the requirements of ASTM C 33.
- o Air-entraining admixture: Material shall conform to ASTM C 260.
- o Water-reducing admixture: Material shall conform to ASTM C 494.
- o Pozzolan: Pozzolan shall conform to ASTM C 618.

Precast concrete mixture shall be designed to satisfy the following requirements. Prior to commencing operations, the Contractor shall furnish the proportions of all ingredients that will be used in the manufacture of precast concrete panel elements.

Concrete mixture proportions shall be selected to satisfy the following requirements. The proportions may be based on past field experience or on trial mixtures in accordance with ACI 318, paragraphs 4.2 and 4.3. Contractor shall provide data demonstrating that the proposed mixture satisfies the following requirements:

- o Minimum 28-day compressive strength: 6,500 psi
- o Maximum coarse aggregate size: 3/4 in. nominal
- o Minimum entrained air: 5 to 7%
- o Maximum water cement ratio: 0.40
- o Minimum cement content: 540 lb/cy (6-sack mixture)
- o High range water reducer (maximum). 5 oz/100 lb cement

b. Reinforcing: Mild steel reinforcing shall be new billet steel bar conforming to the requirements of ASTM A 615, Grade 60.

c. Ties: Weldable grade reinforcing steel conforming to the requirements of ASTM A 706, Grade 60, shall be used for form ties.

d. Prestressing strand: Prestressing strand, if used, shall conform to the requirements of ASTM A 416, Grade 270.

e. Welded wire reinforcement: Welded smooth wire fabric shall conform to the requirements of ASTM A 185.

f. Nonshrink grout: Grout for horizontal construction joints shall be a prepackaged, cementitious based/natural aggregate, nonshrink grout. Mixing water shall be added in accordance with the manufacturer's recommendations to obtain a plastic consistency which will level off and redistribute under the pressure of the upper precast panel.

2.2.2 <u>Accessories</u>

a. Hardware and accessories shall be incorporated into the work as shown on the drawings. Hardware and accessories shall be fabricated in accordance with Section 4.

b. Epoxy grout shall be Concressive 1463 as manufactured by Adhesive Engineering Co., Carlstadt, NJ, or equal. Epoxy grout shall be mixed and applied in accordance with epoxy manufacturer's recommendations.

c. Form ties shall be capable of withstanding the full anticipated load of concrete infill placement with a safety factor of at least 3.0 on failure of the tie. Ties shall be anchored to monolith concrete using epoxy grout or polyester resin cartridge anchors.

d. Bearing pads and horizontal joint seals shall be preformed neoprene material of the size, dimensions, and characteristics shown on the drawings. Vertical joint seals shall be an asphalt-impregnated, open-cell foam.

2.2.3 Fabrication

a. Precast panels shall be fabricated to the dimensions shown on the drawings. Dimensional tolerances shall not exceed in those specified in Section 2.2.4.

b. All reinforcing, inserts, hardware, and appurtenances shall be located as required and securely anchored to prevent movement during concrete placement.

c. Contractor shall moist-cure precast panels until the concrete reaches a minimum strength of 0.7 f'_c. Precast concrete panels may be steam cured. Control of concrete temperature during the steam cycle shall be maintained per the guidelines of PCI MNL-116. Membrane curing compound shall be applied to the outside surface of the panel after completion of the moist-curing cycle.

d. Panels shall not be erected until concrete strength has reached 6500 psi.

2.2.4 Tolerances

a. Dimensional Tolerances for Precast Panel Fabrication

- o Length: ±1/2 in.
- o Width: ±1/4 in.
- o Thickness: ±1/2 in.
- o Edge squareness: ±1/8 in.
- o Planeness (measured with respect to a straight line drawn between any two opposite edges)
  - Outside surface: ±1/4 in.
  - Inside surface: 1/2 in.
- o Location of embedments: ±1/4 in.

b. Location Tolerance for Precast Panel Erection

- o Plumbness or vertical alignment: ±1/2 in.

- o Variation in horizontal alignment: ±1/2 in.
- o Precast element joint to joint alignment
  - Horizontal joints: ±1/8 in.
  - Vertical joints: ±1/8 in.

2.2.5 Finishes

Precast panels shall have a smooth dense finish on the outside, exposed surface such as is typical of steel form or high density overlaid plywood forms. The inside panel surface shall be clean, free of laitance, and shall be intentionally roughened to an approximate amplitude of 1/4 in. The surface shall be cleaned by high-pressure water spray immediately prior to erection.

2.3 EXECUTION

2.3.1 Preparation

a. Contractor shall inspect and survey all existing work prior to fabricating and installing panels. Dimensional discrepancies shall be immediately brought to the attention of the Engineer.

b. Contractor shall prepare a written erection procedure indicating lifting, temporary bracing, support and alignment methods. Sequence of operations and inspection hold points shall be identified.

2.3.2 Erection

a. Precast panels shall be erected as shown on the drawings. Tolerances shall be as specified in Section 2.2.4.

b. Panel form ties shall be securely fastened. Temporary supports and braces shall be used as necessary to maintain alignment.

c. Nonshrink grout joint sealant, bearing pads, and neoprene seals shall be installed and applied as required on the drawings. Care shall be taken to prevent seals from being displaced during panel installation.

d. Form tie holes shall be drilled into monolith using suitable concrete drilling equipment. Ties shall be installed and set using epoxy grout or polyester cartridge anchors. Form tie grout shall be allowed to set for a minimum of 24 hours prior to erecting precast panels. Form ties shall be embedded and grouted to develop a minimum ultimate tensile load of 33 kips when tested in accordance with Section 2.3.3.f.

2.3.3 Inspections and Tests

a. Contractor shall be responsible for inspection of panel fabrication and erection activities to ensure that the work conforms in all respects to the drawings and specifications. Record of inspections and inspection results shall be maintained for Engineer's review.

b. Contractor shall inspect precast panel form prior to casting to ensure dimensional configuration and location of all embedded items.

c. Contractor shall inspect panels after installation to ensure conformance to location tolerance and that panels are securely tied and braced for infill concrete placement loads.

d. Contractor shall sample and test concrete used in panel fabrication as follows:

   o A minimum of two sets of three concrete specimens shall be cast, cured, and tested for each batch of concrete to determine the concrete compressive strength at 7 and 28 days. Contractor may make additional specimens to monitor strength gain during cure.

   o Determine slump of concrete mixture at time of placement.

   o Determine air content at time and point of placement.

e. Contractor shall provide to the Corps or the Engineer, for independent testing and analysis, such additional cast specimens of concrete as may be requested.

f. Contractor shall install a minimum of two additional form ties and two additional dowels at a convenient location in the monolith as directed by the Engineer. Contractor shall conduct tensile testing of the installed ties and dowels to determine the minimum ultimate load of the installed ties and dowels. The minimum ultimate load of the tie or dowel shall be that load at which it ruptures, slips excessively, or exhibits greater than 1/4 in. of total deflection between the face of the monolith and the connection to the form. The average of the two tension tests shall be used to establish the ultimate load of the ties and dowels.

2.3.4 Cleanup

a. Contractor shall immediately remove spills or runs of epoxy, grout, or other materials used in the construction from the outside surface of precast panels.

b. Contractor shall maintain work areas clean and free of rubble, discarded product containers, packaging and shipping materials and other refuse.

SECTION 3
CAST-IN-PLACE CONCRETE

3.1 GENERAL

3.1.1 Work Included

The work includes all materials and workmanship required for production, delivery, placing, and curing of cast-in-place concrete.

3.1.2 Related Work

Related work includes fabrication of embedded lock hardware and appurtenances, and fabrication and erection of precast concrete leave-in-place form elements.

3.1.3 References

a. ACI 301: Structural Concrete for Buildings
b. ACI 318: Building Code Requirements for Reinforced Concrete

3.1.4 Submittals

a. Contractor shall submit concrete mixture proportions and test results as specified in Paragraph 3.2.1.a, below.

b. Contractor shall submit batch monitoring test results for infill concrete placements as required by Paragraph 3.3.3.b, below.

3.1.5 Quality Control

a. All work shall be performed in accordance with ACI 301 and 318 as applicable.

b. Contractor shall maintain records of tests and inspections as required herein and make copies of such records available to Engineer on request.

3.2 PRODUCTS

3.2.1 Materials

a. Concrete: Cast-in-place concrete materials shall conform to the following requirements:

- o Cement: Cement shall conform to the requirements of ASTM C 150.

- o Aggregate: Aggregate shall conform to the requirements of ASTM C 33.

- o Air-entraining admixture: Material shall conform to ASTM C 260.

- o Water-reducing admixture: Material shall conform to ASTM C 494, Type E.

- o Pozzolan: Pozzolan shall conform to ASTM C 618.

Cast-in-place concrete mixture shall be designed to satisfy the following requirements. Prior to commencing operations, the Contractor shall furnish the proportions of all ingredients that will be used in the manufacture of cast-in-place concrete. The mixture proportions shall be accompanied by test results from an independent commercial testing laboratory, attesting that the proportions selected will produce concrete of the required quality.

- o Minimum 28-day compressive strength: 3000 psi

- o Maximum coarse aggregate size: 3/4 in. nominal

- o Minimum entrained air: 3% to 5%

- o Maximum water cement ratio: 0.5

- o Minimum cement plus fly ash content: 450 lb/cy (5-sack mixture)

- o High range water reducer: Type F

b. Reinforcing: Mild steel reinforcing shall be new billet steel bar conforming to the requirements of ASTM A 615, Grade 60.

3.2.2 Accessories

a. Hardware and accessories shall be incorporated into the work as shown on the drawings. Hardware and accessories shall be fabricated in accordance with Section 4.

b. Joint seals and joint filler materials shall be as shown on the drawings.

3.3 EXECUTION

3.3.1 Preparation

a. Contractor shall prepare a written concrete placing procedure, identifying placing sequence, maximum allowable lift height and placing rate, and including a checklist of inspections to be made. Contractor's proposed methods of concrete placement, consolidation, and curing shall be submitted to the Engineer for review.

b. Contractor shall inspect all existing work prior to placing cast-in-place concrete to verify that such work is complete and ready to receive concrete.

c. Contractor shall notify Engineer at least 24 hours in advance of any concrete placement.

d. All embedded items and reinforcement shall be securely tied to prevent movement during concrete placement and consolidation activities.

e. Formwork for cast-in-place concrete shall be in accordance with ACI 301.

3.3.2 Concrete Mixing, Placing, and Curing

Production, conveying, placing, consolidation, and curing of concrete shall be performed in accordance with ACI 301, and of the following requirements:

a. Truck mixers may be used with written approval of the Engineer. When admixtures are dispensed into the truck at the site, truck capacities and batch sizes shall be selected that enable thorough and complete mixing of all constituent materials.

b. All exposed concrete surfaces shall be cured by application of absorptive mats or fabric kept continuously wet for a minimum of seven days after concrete placement. Membrane curing shall not be used.

c. Contractor shall measure precast panel movements and deflections before and after placing concrete. The measurements shall be taken at all four panel corners and at the top, bottom, and midheight along a vertical line passing through the middle of the panel. These measurements shall be made for Panels P-1, P-2, and P-7. The Contractor shall submit his proposed measuring procedures to the Engineer for review.

3.3.3 Inspections and Tests

a. The Contractor shall be responsible for inspection and testing of cast-in-place concrete activities to ensure that the work conforms in all respects to the drawings and specifications. Records of inspections and tests shall be maintained for Engineer's review.

b. The Contractor shall provide the following necessary quality control and testing services for each batch of concrete placed.

- Two sets of three concrete specimens shall be cast, cured, and tested to determine the concrete compressive strength at 7 and 28 days.
- Determine slump of concrete mixture at time of placement.
- Determine air content of concrete mixture at time and point of placement.

3.3.4 Cleanup

Contractor shall dispose of wasted concrete in accordance with Corps' requirements. Mixer trucks, pumps, tools, and placing equipment shall be cleaned in designated areas only, and wash water and spoil shall be contained as required.

SECTION 4
HARDWARE AND APPURTENANCES

4.1 GENERAL

4.1.1 Work Included

a. The work shall consist of furnishing all labor, materials, and equipment for fabrication and furnishing of hardware and appurtenances for the navigation lock repair mock-up demonstration, as shown on the drawings and as described in the specifications.

b. Hardware and appurtenances include

- o Horizontal armor
- o Vertical armor
- o Line hook
- o Top curb armor
- o Panel joint assemblies
- o Panel alignment assemblies
- o Form ties

4.1.2 Reference Standards

a. AISC "Manual of Steel Construction", Eighth Edition

b. AWS D1.1-85, "Structural Welding Code"

c. SSPC-SP-6 Steel Structures Painting Council, "Commercial Blast Cleaning"

4.1.3 Quality Control

a. Fabricator Qualifications: The fabricator shall be experienced in the fabrication and working of metals, including cutting, bending, forming, welding, and finishing. Fabrication of metal hardware and appurtenances shall be performed in accordance with the AISC Code.

b. Welder Qualifications: Fabricators supplying welded components shall employ only welders, operators, and tackers qualified as outlined in AWS D1.1. Welding practices shall conform to AWS D1.1.

4.1.4 Submittals

a. Contractor shall submit complete shop drawings indicating all shop and erection details including materials of construction, finishes, methods of fastening, and location of cuts, copes, connections, holes, fasteners, and welds.

b. Contractor shall submit certificates of welder's qualifications prior to start of work of this section.

c. Contractor shall submit mill certificates for structural steel, indicating specification compliance for chemical properties, tensile strength, yield point, and elongation.

d. Contractor shall submit catalog cuts and certificates of compliance for concrete anchors, fasteners, headed studs, and other commercial products incorporated into the work.

4.2 PRODUCTS

4.2.1 Steel

a. Structural steel shapes, plates, and bars shall conform to ASTM A 36, unless noted otherwise.

b. Steel pipe shall conform to ASTM A 53, Type E or S, Grade B, unless noted otherwise.

4.2.2 Bolting Materials and Fasteners

a. Bolting material shall be either ASTM A 307 or A 449 as shown on the drawings. Bolts shall be furnished with matching nuts and washers.

b. Headed studs shall conform to ASTM A 108.

c. Deformed bar anchors shall conform to ASTM A 496.

4.2.3 Other Materials

All other materals not specifically described but required for a complete and proper installation shall be as selected by the contractor subject to approval by the Engineer.

4.3 EXECUTION

4.3.1 Fabrication

a. All structural and miscellaneous steel for hardware and appurtenances shall be fabricated in accordance with the reviewed shop drawings and shall conform to the requirements of the AISC "Manual of Steel Construction."

b. Welding of steel hardware and appurtenances shall conform to AWS D1.1, "Structural Welding Code." Type, size, and spacing of welds shall be as indicated on the reviewed shop drawings. Welding shall be accomplished in a manner which will minimize distortion of the finished parts. Weld splatter and oxides on finished surfaces shall be removed. Unless otherwise noted, headed studs and deformed bar anchors shall be welded using automatically timed stud welding equipment. The Contractor shall perform tests, as recommended by the welding equipment manufacturer, to verify proper operation and settings of the welding equipment.

4.3.2 Surface Preparation and Protective Coatings

a. After fabrication, all steel surfaces shall be blast cleaned in accordance with SSPC-SP-6, Commercial Blast Cleaning.

b. Iron and steel surfaces to be embedded in concrete in the completed work shall be uncoated.

c. All exposed surfaces of hardware and appurtenances shall be given a shop coat of zinc-rich, rush-inhibitive primer. The dry film thickness of the primer shall be 2 mils minimum. The steel surface shall be prepared, and the primer applied in accordance with the coating manufacturer's recommendations.

d. Steel surfaces to be uncoated shall be free of loose rust, mill scale, oil, and grease.

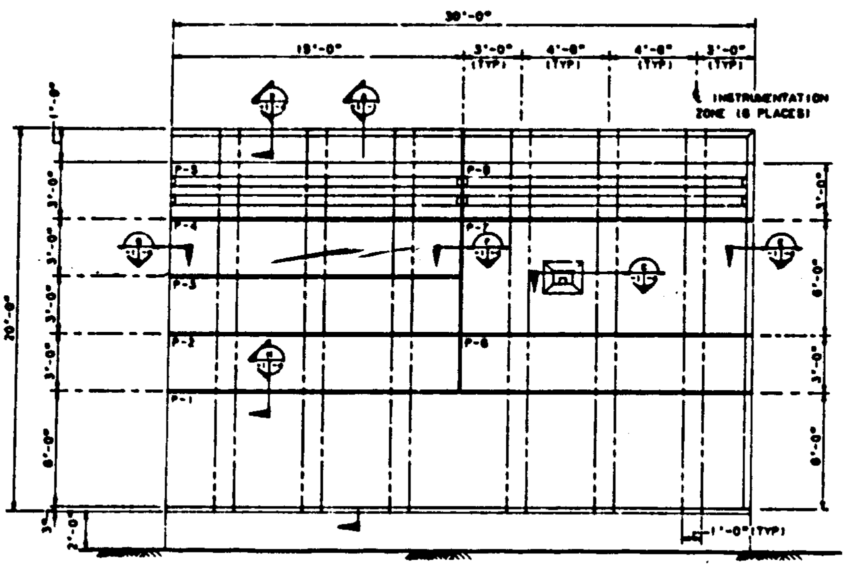
4.3.3 Inspection and Testing

a. Contractor shall be required to perform such inspections and tests to ensure that all work is performed in full compliance with the contract documents.

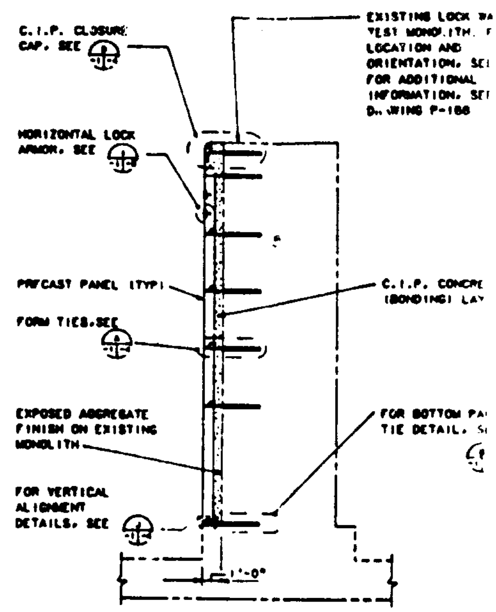
b. Inspection and testing of welding shall be in accordance with AWS D1.1, Section 6.

c. Material and workmanship will be subject to inspection by the Engineer. Testing and inspection by the Engineer will in no way relieve the Contractor of its responsibility to furnish materials and construction in full compliance with the contract documents, and to provide its own quality control program.

APPENDIX C: DESIGN DRAWINGS
PHASE II DEMONSTRATION PROJECT



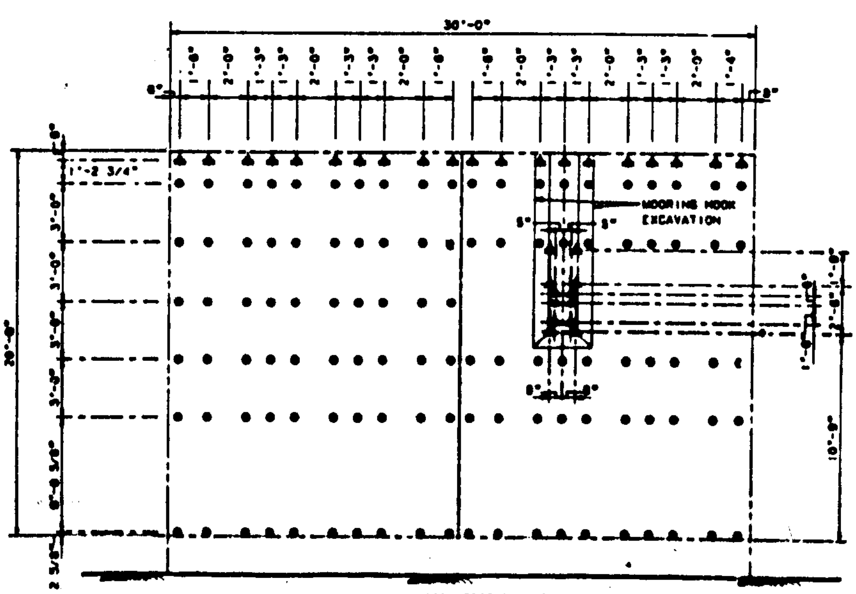
ELEVATION - LOCK SURFACE REHABILITATION MOCK-UP
SCALE: 1/4"=1'-0"



SECTION - LOCK SURFACE REHABIL

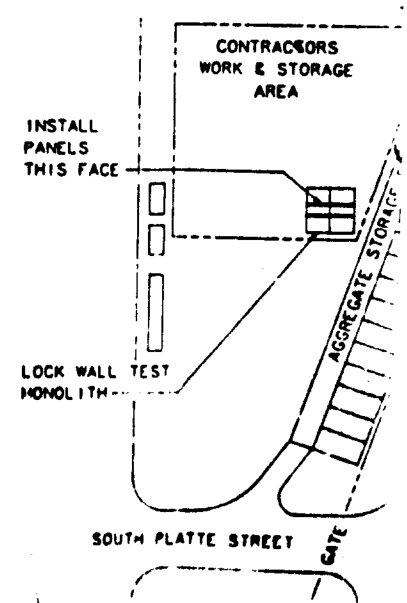
LEGEND:

- ● FORM TIE, SEE
- ▲ 6 BAR DOWELS, SEE



NOTE: TEST FORM TIES AND DOWELS ARE NOT SHOWN. SEE NOTE 8.

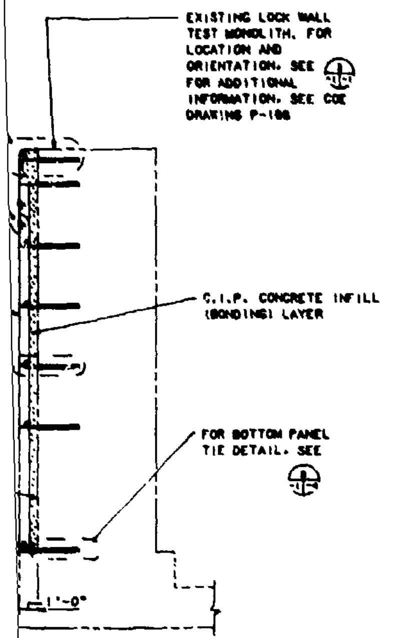
ELEVATION - FORM TIE, DOWEL & CONCRETE EXCAVATION LOCATIONS
SCALE: 1/4"=1'-0"



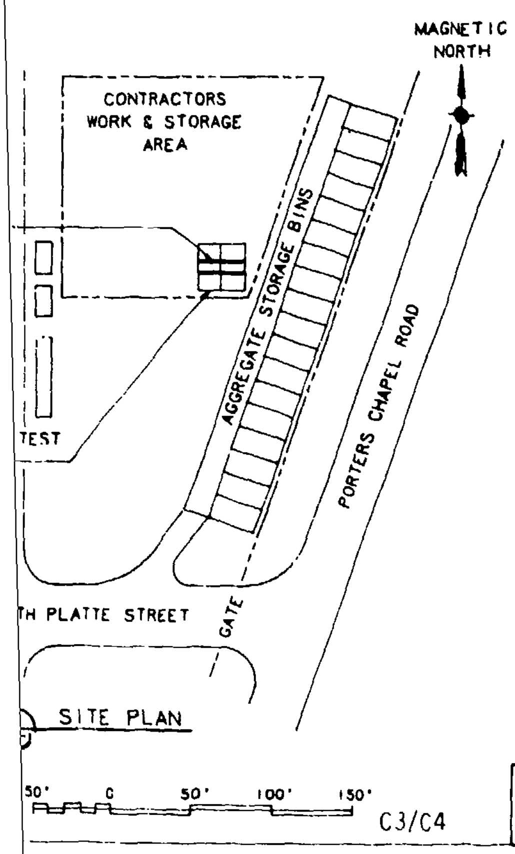
SITE PLAN

1"=50'-0" 50' 0 50' 100

Preceding Pages Blank



LOCK SURFACE REHABILITATION MOCK-UP



SITE PLAN

C3/C4

NOTES

1. THIS WORK WILL DEMONSTRATE THE FEASIBILITY OF REPAIRING DETERIORATING NAVIGATIONAL LOCK WALL SURFACES USING PRECAST CONCRETE PANELS AS STAY-IN-PLACE FORMS. THIS WORK INCLUDES, BUT IS NOT LIMITED TO, CASTING PRECAST CONCRETE PANELS, FABRICATING MISCELLANEOUS STEEL HARDWARE ASSEMBLIES FOR INCORPORATION INTO THE WORK, SURFACE PREPARATION AND MISCELLANEOUS CONCRETE REMOVAL FROM THE MOCK-UP TEST MONOLITH, PANEL ERECTION, AND PLACEMENT OF THE CIP INFILL CONCRETE BONDING LAYER BETWEEN THE MONOLITH AND PRECAST PANELS. A COMPLETE DESCRIPTION OF THE WORK AND REQUIREMENTS ARE CONTAINED IN THE SPECIFICATIONS.

2. THE TEST MONOLITH IS BEING PROVIDED BY THE CORPS OF ENGINEERS (COE) AND IS LOCATED AT THE WATERWAYS EXPERIMENT STATION (WES) AS SHOWN IN THE SITE PLAN. ALL WORK PERFORMED ON COE'S PREMISES SHALL BE IN ACCORDANCE WITH COE RULES AND REGULATIONS. THE CONTRACTOR'S ON-SITE WORK AND LAYDOWN AREAS ARE SHOWN ON THE SITE PLAN. THE CONTRACTOR SHALL ARRANGE FOR ANY ADDITIONAL OFF-SITE FACILITIES IF REQUIRED.

3. THE CONTRACTOR SHALL SUBMIT A PRELIMINARY WORK SCHEDULE 2-WEEKS PRIOR TO COMMENCING WORK. SUBSEQUENT TO REVIEW BY COE, THE SCHEDULE WILL BE FINALIZED TO REFLECT TIME REQUIRED FOR COE TO INSTALL ANY INSTRUMENTATION OR TO COORDINATE ANY TESTING.

4. WORK SEQUENCE:

   A. SURFACE PREP, DRILL HOLES, INSTALL DOWELS/TIES
   B. ERECT PANEL P-1, PLACE CIP INFILL CONCRETE FOR 15-FT WIDTH COINCIDENT W/ PANELS P-2 TO P-5
   C. ERECT PANEL P-2, PLACE CIP INFILL CONCRETE
   D. ERECT PANEL P-3, PLACE CIP INFILL CONCRETE
   E. ERECT PANEL P-4, PLACE CIP INFILL CONCRETE
   F. ERECT PANEL P-5, PLACE CIP INFILL CONCRETE
   G. ERECT PANELS P-6 TO P-8, PLACE INFILL CONCRETE FULL 28-FT HEIGHT OF WALL
   H. COMPLETE CONCRETE CAP CONSTRUCTION
   I. REMOVE AUXILIARY FORMING AND TIES, PATCH TIE HOLES

5. THE PRECAST PANELS AND FORM TIES HAVE BEEN DESIGNED FOR A MAXIMUM PRESSURE OF 1.25 KSF. PLACEMENT RATES FOR THE CIP INFILL CONCRETE SHALL BE CONTROLLED SO AS NOT TO EXCEED THE DESIGN PRESSURE.

6. MATERIALS

PRECAST CONCRETE:	f'c = 5000 PSI, W/C RATIO = 0.40
CIP CONCRETE:	f'c = 3000 PSI, W/C RATIO = 0.50
REINFORCING STEEL:	ASTM A 615, GRADE 60
FORM TIES (REINF):	ASTM A 706, GRADE 60
WELDED WIRE FABRIC:	ASTM A 185
HEADED WELD STUDS:	ASTM A 108
DEFORMED BAR ANCHORS:	ASTM A 496 Fy = 70 KSI
STEEL FOR HARDWARE:	ASTM A 36 UNLESS OTHERWISE NOTED
OTHER MISC MATERIALS:	AS NOTED IN THE DRAWINGS

STEEL FABRICATION: AISC MANUAL OF STEEL CONSTRUCTION

WELDING: AWS D1.1-85 STRUCTURAL WELDING CODE. USE AUTOMATICALLY TIMED STUD WELDING EQUIPMENT FOR WELD STUDS AND DEFORMED BAR ANCHORS UNLESS OTHERWISE NOTED.

CONCRETE: ACI 301 STRUCTURAL CONCRETE FOR BUILDINGS
ACI 318 BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE
PCI MNL-116 MANUAL FOR QUALITY CONTROL FOR PLANTS AND PRODUCTION OF PRECAST PRESTRESSED CONCRETE PRODUCTS

8. NO DRILLING, CHIPPING, OR CUTTING OF CONCRETE IS PERMITTED WITHIN THE INSTRUMENTATION ZONES IDENTIFIED ON THE DRAWINGS WITHOUT SPECIFIC WRITTEN APPROVAL OF THE COE. DEMOLITION/EXCAVATION OF CONCRETE FOR THE LINE HOOK RECESS SHALL BE ACCOMPLISHED WITHOUT DISTURBING CONCRETE AT THE ADJACENT INSTRUMENTATION ZONES. DEMOLITION TOOLS SHALL BE SIZED TO ENABLE CONTROLLED BREAKAGE AND REMOVAL OF CONCRETE.

9. TENSILE TESTING OF SAMPLE FORM TIES AND REBAR DOWELS, AS NOTED IN THE SPECIFICATIONS, SHALL BE CONDUCTED TO ESTABLISH THE REQUIRED EMBEDMENT DEPTH. THE EMBEDMENT DEPTH SHALL BE SUFFICIENT TO DEVELOP A MINIMUM FORCE OF 33 KIPS. HOLES FOR FORM TIES AND REBAR DOWELS SHALL BE DRILLED WITH ROTARY-PERCUSSIVE EQUIPMENT. HOLES SHALL BE THOROUGHLY CLEANED PRIOR TO INSTALLING BONDING MATERIAL AND TIES/DOWELS. THE CONTRACTOR MAY ELECT TO INCLINE DOWEL AND TIE HOLES UP TO 15-DEGREES MAXIMUM TO ASSIST IN RETAINING BONDING MATERIAL.

10. CONTRACTOR SHALL PERFORM AS-BUILT MEASUREMENTS OF EXISTING LOCK-WALL TEST MONOLITH AND MAKE THE NECESSARY ADJUSTMENTS TO THE NEW WORK TO SUIT EXISTING WORK AND ADVISE ENGINEER OF MAJOR DISCREPENCIES.

11. TITLE SYMBOLS

INDICATED REFERENCED VIEW NUMBER = PLAN, ELEVATION, DETAIL LETTER = SECTION

SHEET NUMBER WHERE VIEW IS INITIALLY IDENTIFIED

SHEET NUMBER WHERE VIEW IS DRAWN

SYM	DESCRIPTION	DATE	APPROVED
0	FOR CONSTRUCTION		

REVISIONS

A86029

E. OZOLIN

D. KOSKI

D. MAGURA

ABAM CONSULTING ENGINEERS

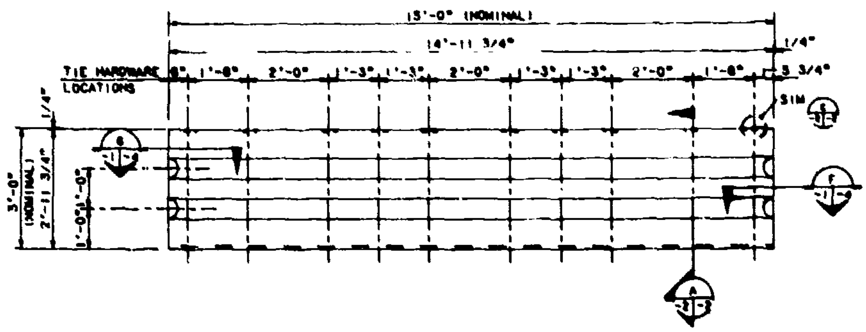
GENERAL ARRANGEMENT

U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION

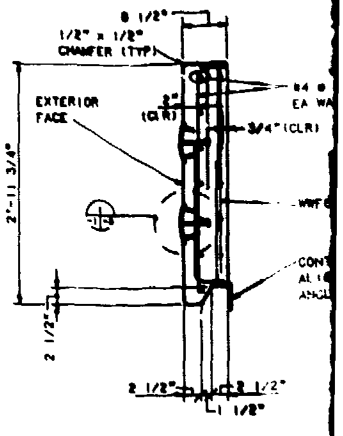
CORPS OF ENGINEERS

A86029-1 0

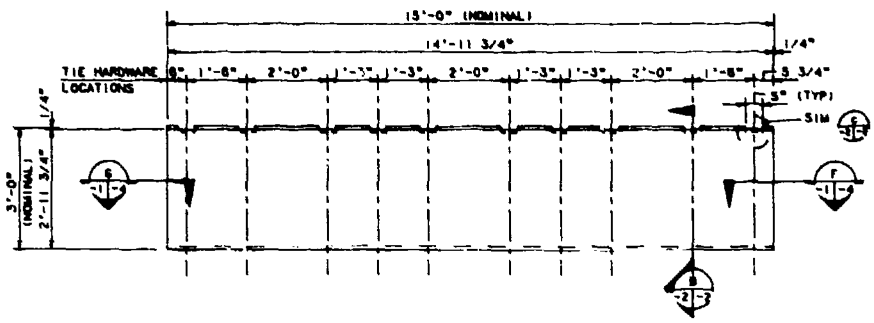
R. MOHN | 11 JULY 86 | SCALE: AS NOTED | SHEET 1 OF 5



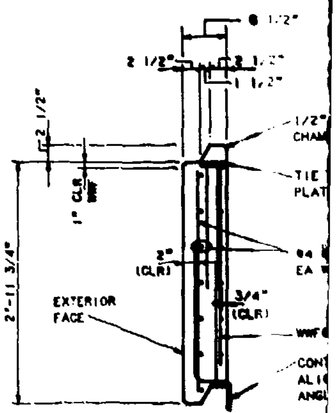
ELEVATION - PRECAST PANEL 'P-5'
SCALE: 1/2"=1'-0"



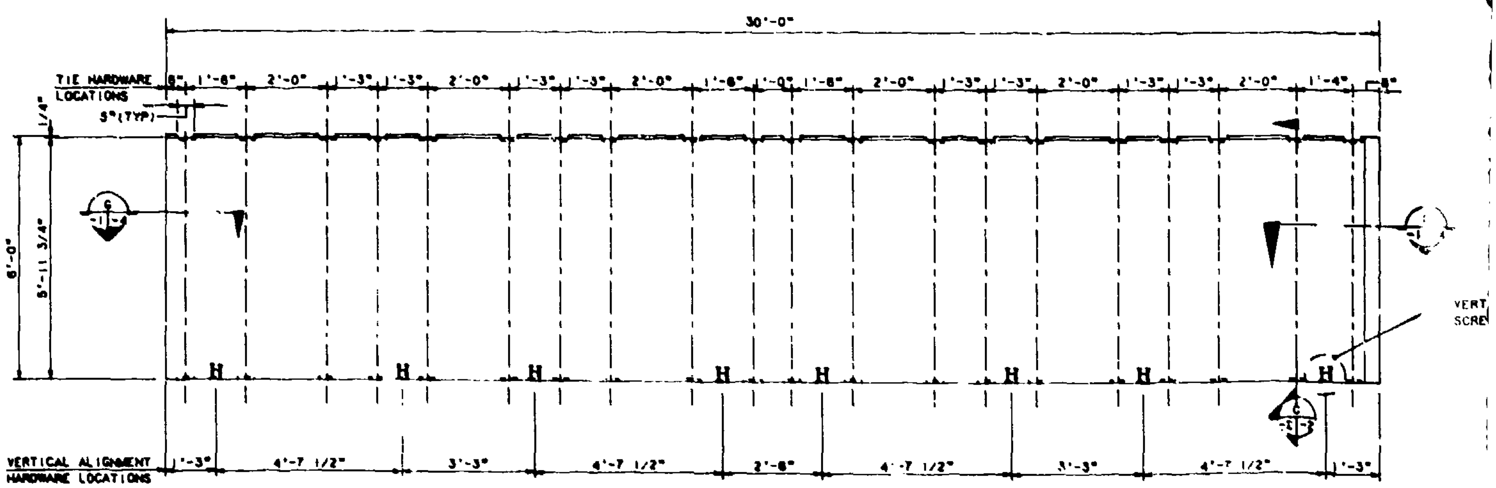
SECTION - PRECAST P
SCALE: 1"=1'-0"
'P-5' &



ELEVATION - PRECAST PANELS 'P-2','P-3','P-4'
SCALE: 1/2"=1'-0"



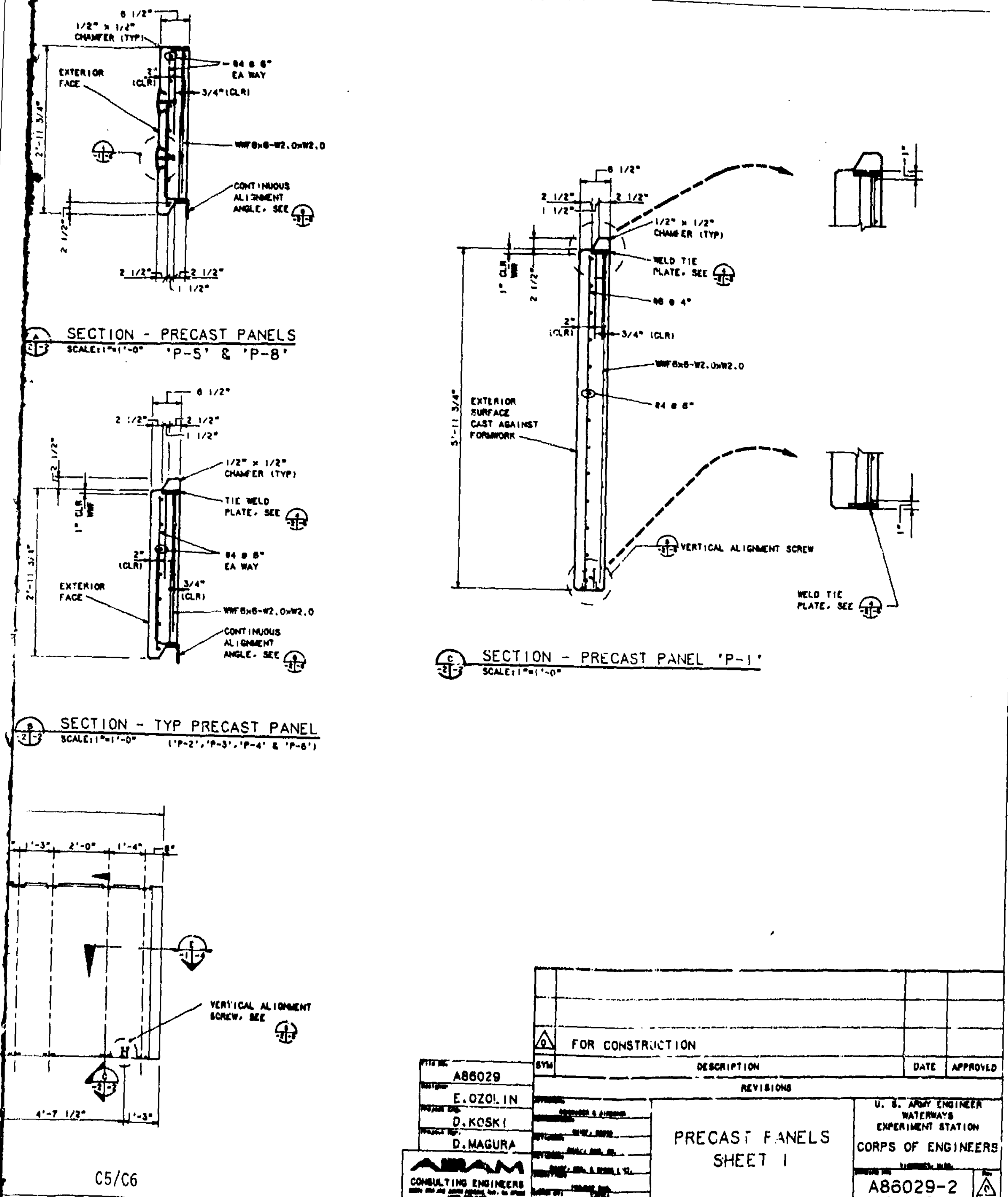
SECTION - TYP PRECA
SCALE: 1"=1'-0"
('P-2' 'P-3',



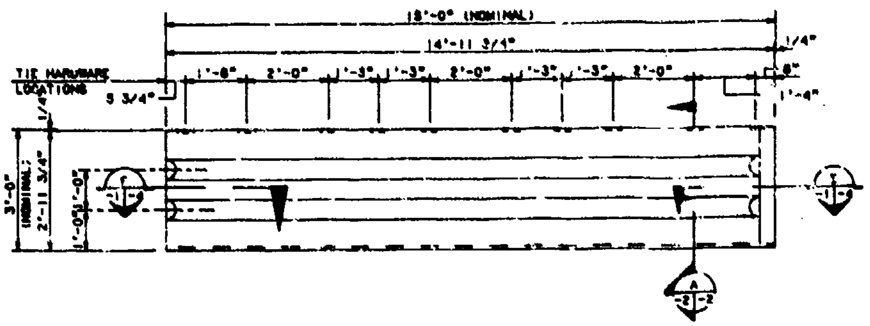
ELEVATION - PRECAST PANEL 'P-1'
SCALE: 1/2"=1'-0"

C5/C6

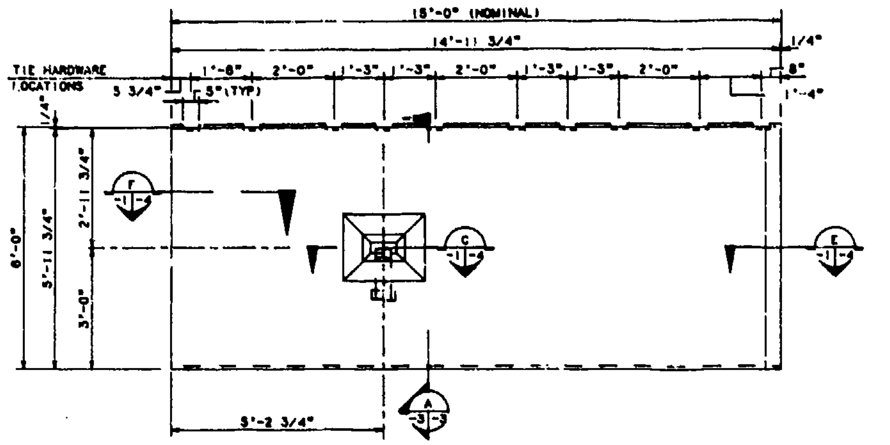
2



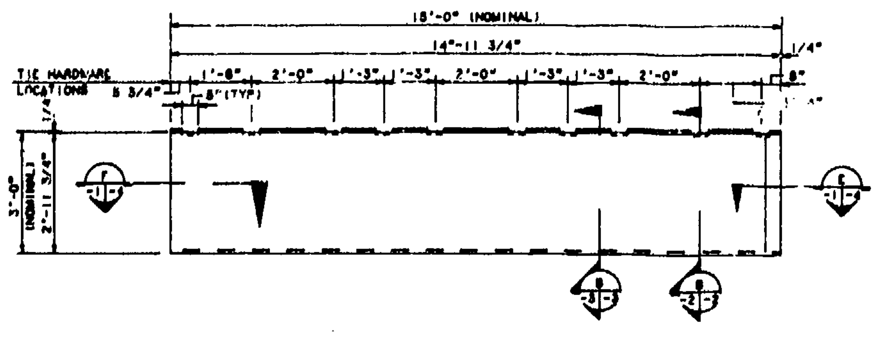
6 1/2"
1/2" x 1/2" CHAMFER (TYP)
#4 @ 8" EA WAY
EXTERIOR FACE
2" (CLR)
3/4" (CLR)
WWF6x6-W2.0xW2.0
CONTINUOUS ALIGNMENT ANGLE, SEE
2'-11 3/4"
2 1/2"
1 1/2"
SECTION - PRECAST PANELS 'P-5' & 'P-8'
SCALE: 1"=1'-0"
TIE WELD PLATE, SEE
1" CLR MIN
SECTION - TYP PRECAST PANEL ('P-2', 'P-3', 'P-4' & 'P-6')
SCALE: 1"=1'-0"
WELD TIE PLATE, SEE
#6 @ 4"
EXTERIOR SURFACE CAST AGAINST FORMWORK
5'-11 3/4"
VERTICAL ALIGNMENT SCREW
SECTION - PRECAST PANEL 'P-1'
SCALE: 1"=1'-0"
1'-3"
2'-0"
1'-4"
8"
VERTICAL ALIGNMENT SCREW, SEE
4'-7 1/2"
C5/C6
FOR CONSTRUCTION
SYM
DESCRIPTION
DATE
APPROVED
REVISIONS
A86029
E. OZOLIN
D. KOSKI
D. MAGURA
CONSULTING ENGINEERS
R. MOHN
11 JULY 88
PRECAST PANELS
SHEET 1
SCALE: AS NOTED
U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION
CORPS OF ENGINEERS
A86029-2
SHEET 2 OF 5



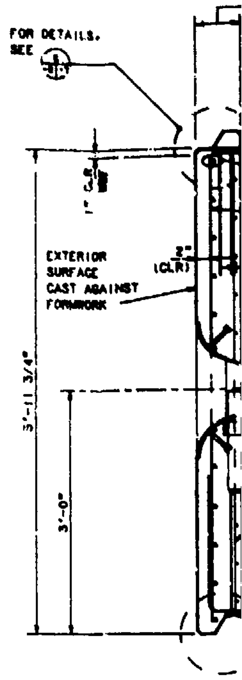
ELEVATION - PRECAST PANEL 'P-8'
SCALE: 1/2"=1'-0"



ELEVATION - PRECAST PANEL 'P-7'
SCALE: 1/2"=1'-0"

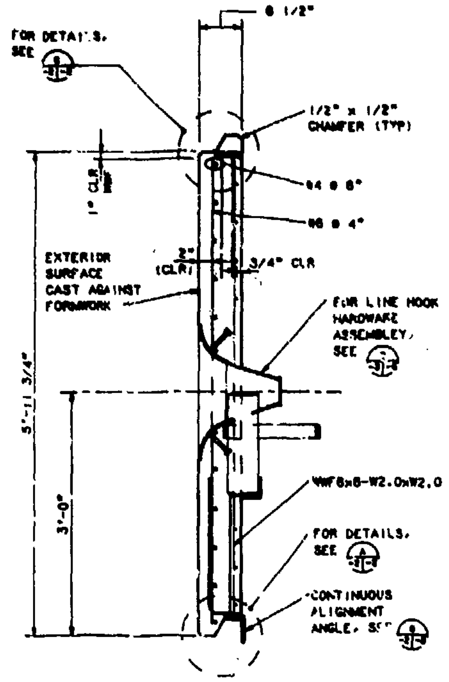


ELEVATION - PRECAST PANEL 'P-6'
SCALE: 1/2"=1'-0"

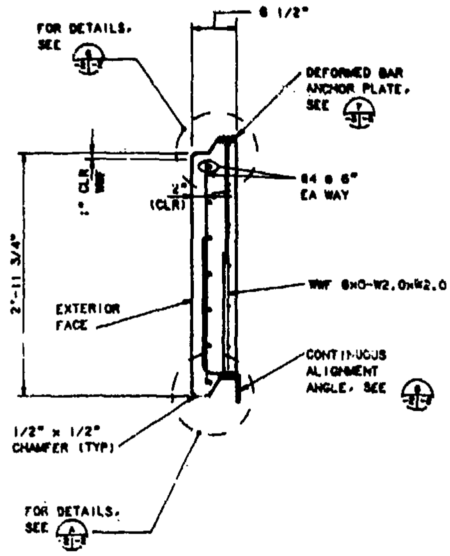


SECTION - PR
SCALE: 1"=1'-0"

C7/C8



SECTION - PRECAST PANEL 'P-7'
SCALE: 1"=1'-0"



SECTION - PRECAST PANEL 'P-6'
SCALE: 1"=1'-0"

SYM	DESCRIPTION	DATE	APPROVED
0	FOR CONSTRUCTION		

REVISIONS

A86029
E. OZOLIN
D. KOSKI
D. MAGURA

ABAM CONSULTING ENGINEERS

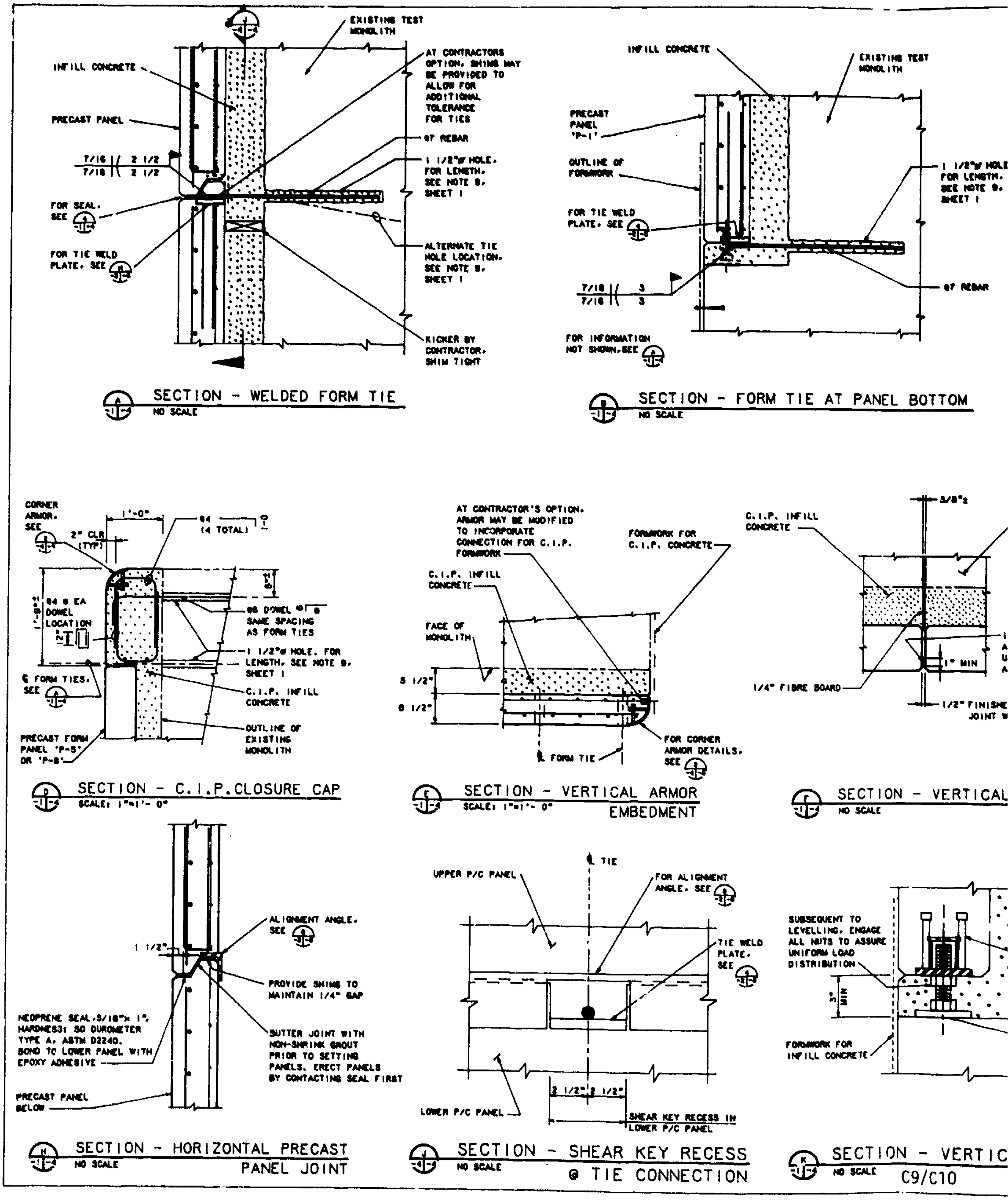
PRECAST PANELS
SHEET 2

U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION
CORPS OF ENGINEERS

A86029-3

R. MOHN | 11 JULY 86 | SCALE: AS NOTED | SHEET 3 OF 5

C7/C8



SECTION - WELDED FORM TIE
NO SCALE

SECTION - FORM TIE AT PANEL BOTTOM
NO SCALE

SECTION - C.I.P. CLOSURE CAP
SCALE: 1"=1'-0"

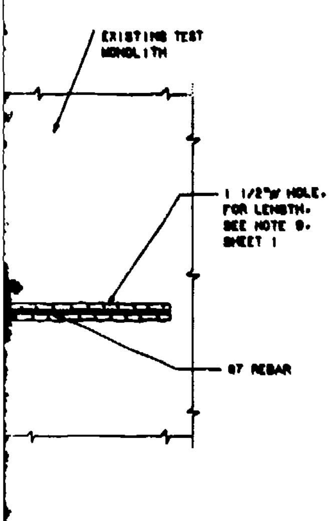
SECTION - VERTICAL ARMOR EMBEDMENT
SCALE: 1"=1'-0"

SECTION - VERTICAL
NO SCALE

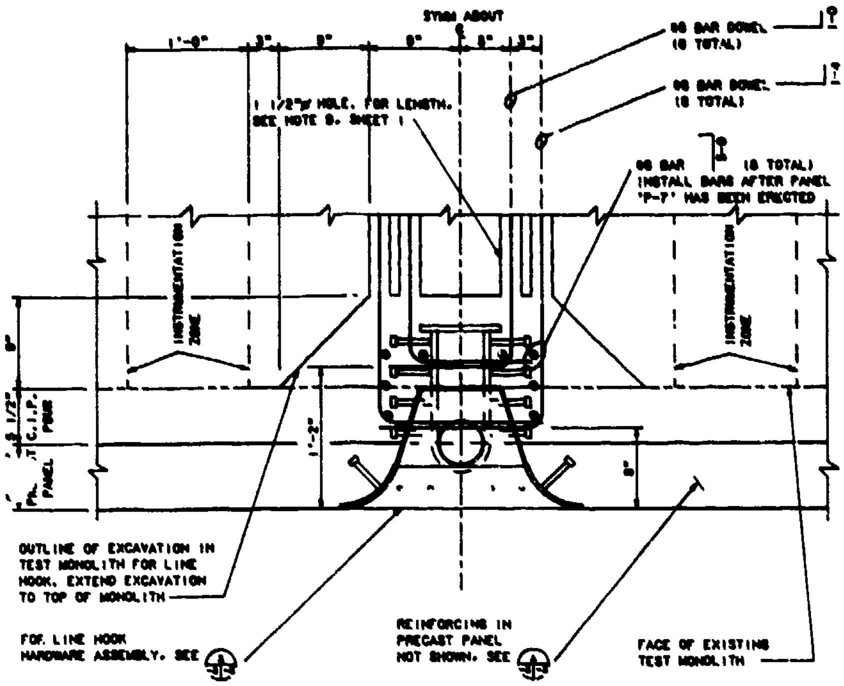
SECTION - HORIZONTAL PRECAST PANEL JOINT
NO SCALE

SECTION - SHEAR KEY RECESS @ TIE CONNECTION
NO SCALE

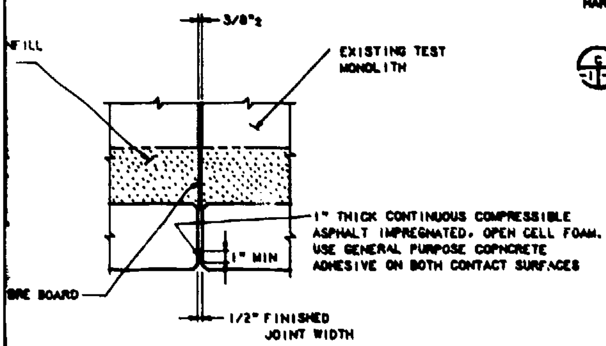
SECTION - VERTIC
NO SCALE C9/C10



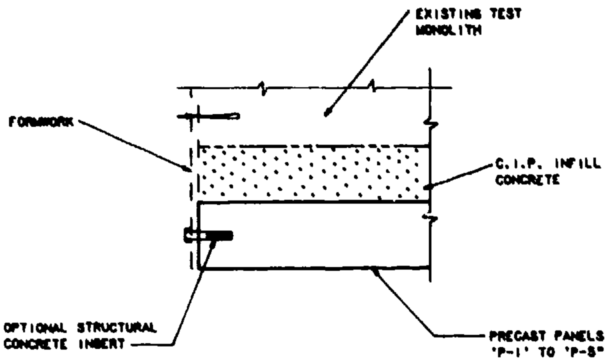
IE AT PANEL BOTTOM



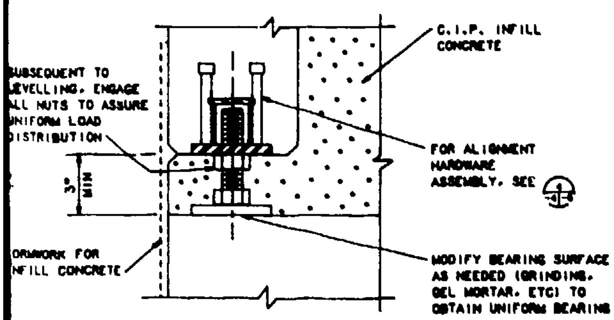
SECTION - SPECIAL EXCAVATION AT LINE HOOK
SCALE: 1-1/2"=1'-0"



SECTION - VERTICAL EXPANSION JOINT
NO SCALE



SECTION - PLAIN END JOINT
NO SCALE



SECTION - VERTICAL ALIGNMENT SCREW
NO SCALE
C9/C10

SYM	DESCRIPTION	DATE	APPROVED
0	FOR CONSTRUCTION		

REVISIONS

A86029
E.OZOLIN
D.KOSKI
D.MAGURA

ABAM CONSULTING ENGINEERS

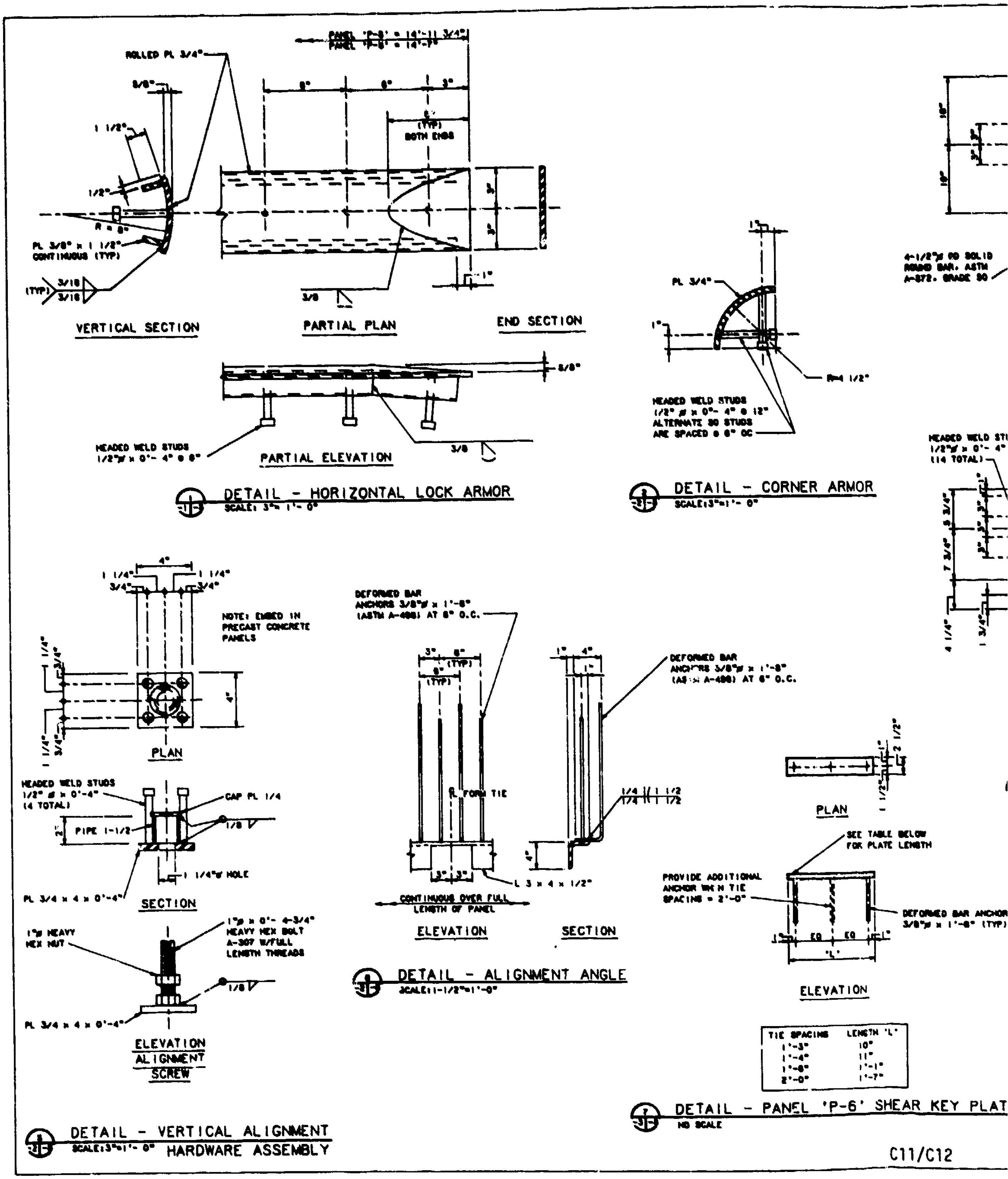
R.MOHN 11 JULY 86

CONCRETE SECTIONS AND DETAILS

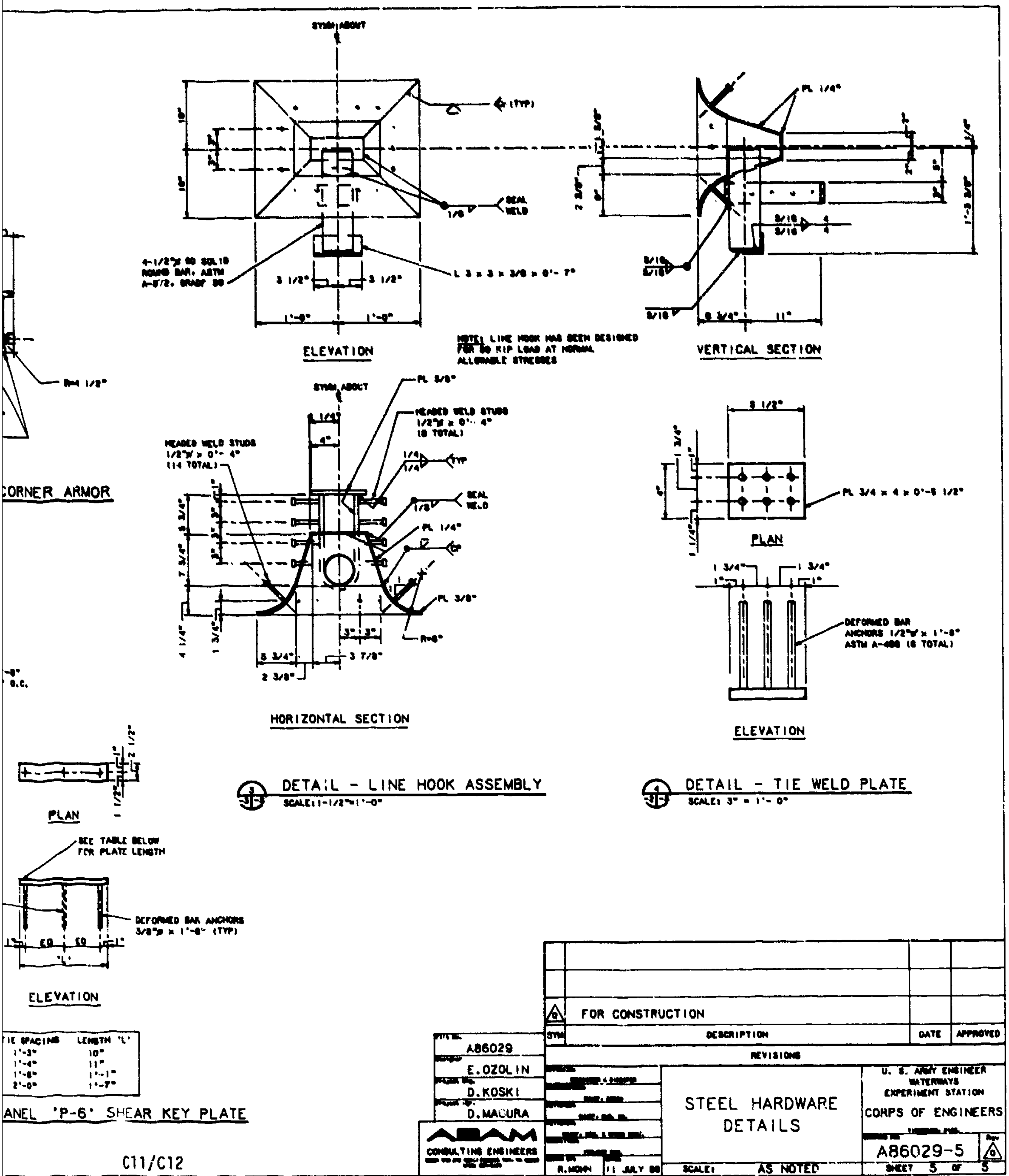
SCALE: AS NOTED

U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION
CORPS OF ENGINEERS

A86029-4
Rev 0
SHEET 4 OF 5



TIE SPACING	LENGTH 'L'
1'-3"	10"
1'-4"	11"
1'-6"	1'-1"
2'-0"	1'-7"



SYM ABOUT ℄
(TYP)
SEAL WELD
1/8
4-1/2"φ 90 SOLID ROUND BAR, ASTM A-572, GRADE 50
3 1/2"
3 1/2"
L 3 x 3 x 3/8 x 0'-7"
1'-0"
1'-0"
ELEVATION
NOTE: LINE HOOK HAS BEEN DESIGNED FOR 50 KIP LOAD AT NORMAL ALLOWABLE STRESSES
PL 1/4"
5/16
5/16
4
4
5/16
5/16
5/16
5 3/4"
11"
2 3/8"
1'-3 3/8"
VERTICAL SECTION
SYM ABOUT ℄
PL 3/8"
HEADED WELD STUDS 1/2"φ x 0'-4" (8 TOTAL)
HEADED WELD STUDS 1/2"φ x 0'-4" (14 TOTAL)
1/4
1/4
TYP
SEAL WELD
1/8
PL 1/4"
CP
PL 3/8"
R=6"
3"
3"
5 3/4"
3 7/8"
2 3/8"
4 1/4"
1 3/4"
7 3/4"
5 3/4"
HORIZONTAL SECTION
3 1/2"
1 3/4"
1"
4"
1 1/4"
PL 3/4 x 4 x 0'-5 1/2"
PLAN
1 3/4"
1 3/4"
1"
1"
DEFORMED BAR ANCHORS 1/2"φ x 1'-6" ASTM A-496 (6 TOTAL)
ELEVATION
3 DETAIL - LINE HOOK ASSEMBLY
SCALE: 1-1/2"=1'-0"
4 DETAIL - TIE WELD PLATE
SCALE: 3" = 1'- 0"
CORNER ARMOR
RM 1/2"
O.C.
2 1/2"
1 1/2"
PLAN
SEE TABLE BELOW FOR PLATE LENGTH
DEFORMED BAR ANCHORS 3/8"φ x 1'-6" (TYP)
1"
EQ
EQ
1"
"L"
ELEVATION
TIE SPACING | LENGTH "L"
1'-3" | 10"
1'-4" | 11"
1'-6" | 1'-1"
2'-0" | 1'-7"
PANEL "P-6" SHEAR KEY PLATE
0 FOR CONSTRUCTION
SYM | DESCRIPTION | DATE | APPROVED
REVISIONS
A86029
E.OZOLIN
D.KOSKI
D.MACURA
ABAM CONSULTING ENGINEERS
R.MOHN
11 JULY 86
STEEL HARDWARE DETAILS
SCALE: AS NOTED
U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION
CORPS OF ENGINEERS
A86029-5
SHEET 5 OF 5

C11/C12